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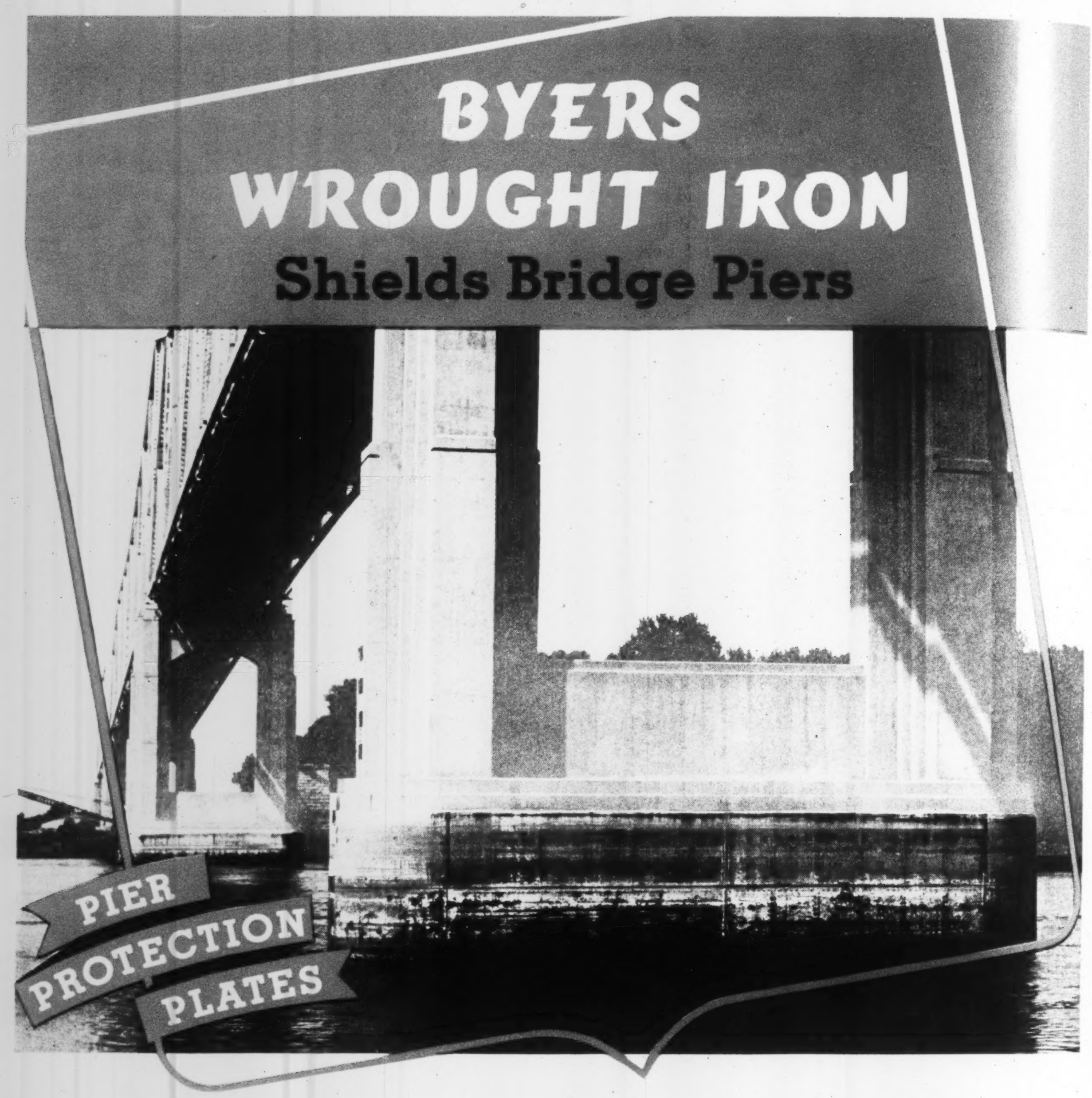

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Editorial and Executive Offices: 330 West 42nd Street, New York . Publication Office: 99-129 North Broadway, Albany, N. Y.

F. E. SCHMITT, *Editor* ALBERT E. PAXTON, *Manager*
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VOL. 120 *MARCH 10, 1938* No. 10

THIS WEEK AND NEXT

REPLENISHMENT of underground water storage, a relatively new water supply problem, is now seasonal routine in the Santa Clara Valley in California where excellent results have been obtained in raising water levels in wells over an area of 130,000 acres.

Two-dimensional stress data are discussed by George E. Beggs and Elmer K. Timby of Princeton University who have been making strain rosette tensometer measurements and deducing design information from them.

By assembling 14 bridge spans on barges and floating them into place, the Marine Parkway Bridge at New York was erected in record time. It is also notable for an open-mesh floor and for containing the longest highway lift span (540 ft.) yet built.

Significant work in concrete investigation, design and practice was reported at the recent meeting of American Concrete Institute in Chicago. Much of this material is abstracted beginning on page 374.

Current problems in highway design and practice were discussed by western road men at the twelfth annual highway engineering conference at the University of Colorado, Boulder, in January.

NEXT WEEK — Roadways on Bridges. Progress in the development of bridge-floor design as exemplified by bridges built in recent years by the Port of New York Authority is to be discussed by E. Warren Bowden, assistant to the chief engineer.

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THE COVER PICTURE is of the record 540-ft. lift span of the Marine Parkway Bridge connecting points on Long Island, New York. American Bridge Co. Photo.

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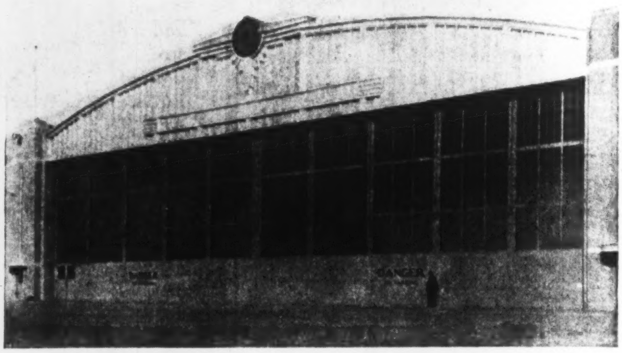

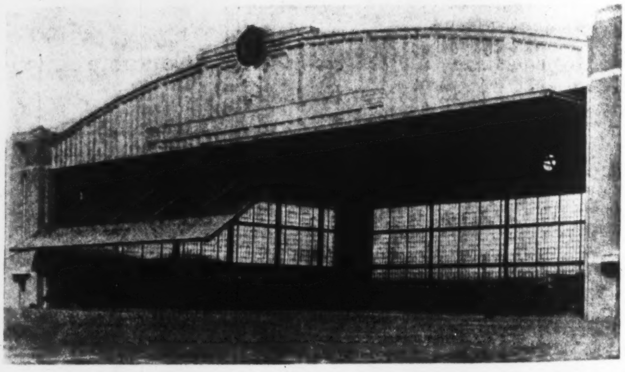

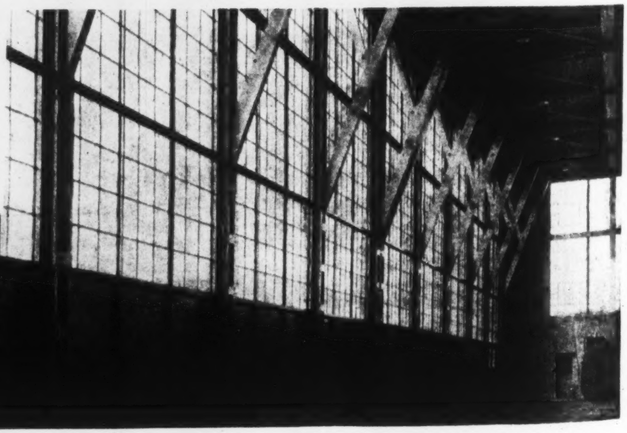

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THE WEEK'S EVENTS

Southern California Coastal Plain Ravaged by Flood

Downpour of 15 in. in single day sends record flood volumes down all stream channels, cutting communication and causing large death list and property loss despite successful functioning of flood-control works so far completed

By N. A. Bowers,
Pacific Coast Editor,
Engineering News-Record

Los Angeles, March 6, 1938—Floods swept the whole Southern California coastal plain area from March 2 to March 5 as a result of an intense rainstorm that covered the full width of the state but which was especially severe in the steep mountain area bordering the Los Angeles-Riverside plain on the north. Despite excellent functioning of the flood-control works so far completed (mainly in Los Angeles County) the floodflow in all streams of the region appears to have exceeded the highest levels known. Railway and wire communication was interrupted and for two days many parts of the flood area were cut off from the outside world. Incomplete figures include a verified death list of 119 and an estimated total list well exceeding 200, and property losses placed well above $50,000,000.

March opened with a five-day storm which brought rain instead of snow in the San Gabriel and San Bernardino mountain areas because of warm air at high altitudes. The flood runoff of the storm filled Los Angeles County's 15 flood-storage reservoirs, swept away old and poorly founded bridges by the score, ate out the approaches to modern bridges at many points, and isolated the Los Angeles metropolitan area for several days, leaving radio and airplane as only means of communication. However, the streams of the region are short and steep and their floods subside quickly; the nearly 10,000 people cared for in schools, churches and refugee camps while their homes were untenable required at most only a few days' aid.

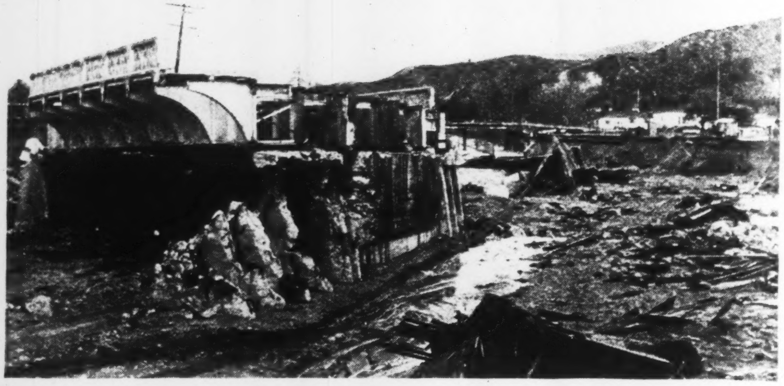
Worst conditions obtained in small communities that suddenly found themselves in the path of raging, debris-laden torrents. Receding waters disclosed wrecked buildings, deep silt and debris in homes, stores and streets, and frequently complete isolation of the community. Anaheim and Fullerton in the path of the Santa Ana River, on which flood protection has not yet started, suffered the highest damage percentage. However, from Ventura south, from ocean to mountains all streams carried destructive floods and no cities along river channels escaped altogether. The degree of damage varied with encroachment on flood channels.

The mountainous areas from which floods discharge on California's populous south coastal plain include 1,200 sq. mi., about half of whose drainage is controlled by reservoirs; the uncontrolled half affects less populous areas. But the flood discharge from these reservoirs requires controlled channels to the sea, and these have been provided only in part. Additional channel revetment and training works on the coastal plain are projected, including two large detention storage basins with embankment dams planned for the Los Angeles River, two for the San Gabriel River and three or four more on smaller streams of the basin.

Rainfall 11 in. in 8 hr.

The recent storm came inland from the sea; rain fell simultaneously on some 2,500 sq. mi. as the clouds moved slowly eastward. The runoff peaks from rainfall on the lowlands largely had passed before the mountain runoff arrived. In coastal plain and foothills the rainfall was 5 to 13 in. for the entire storm; in the mountain area the maxima were about 11 in. in 8 hr. and 15 in. in 24 hr. Along the Owens Valley Aqueduct the maximum was 1 in. per hr. for 2 hr. Inflow at San Gabriel Dam No. 1 reached 100,000 sec.-ft., but peak discharge over the spillway was only 55,000 sec.-ft. and would have been less had the bypass valves been completed. These figures are typical; generally, the flood-control dams decreased peak flows by one-third to one-half, thus

MAIN LINE bridge of the Southern Pacific Ry. across the Los Angeles River in Los Angeles, two spans of which were dropped into the channel when a pier was undermined and overturned.



Remains of La Canada St. Bridge over Verdugo Wash. The channel for Verdugo Wash, built originally with too small a cross-section to carry a large flood, was being enlarged as part of the current flood-control work.

Pedestrian bridge near the mouth of the Los Angeles River in Long Beach that was undermined and partly swept away, causing the loss of several lives.

avoiding enormous additional flood losses.

Estimated stream peaks at the ocean were: Los Angeles River 70,000 sec.-ft.; Ballona Creek 24,000 sec.-ft.; San Gabriel River 50,000; Santa Ana River well over 100,000 sec.-ft. Comparisons with previous maxima are difficult because records of 1884 and earlier are meager and many streams are now controlled. Some authorities rate the recent storm an all-time record.

On the San Gabriel, storm comparisons are possible by noting the partly-controlled 100,000 sec.-ft. inflow into No. 1 reservoir and the previous high record on Jan. 18, 1916, when the canyon-mouth discharge was 40,000 sec.-ft.

Control works effective

The Los Angeles County Flood-Control District reports all fifteen of its dams undamaged. Some of them spilled for the first time; all reservoirs were filled. Big Tujunga No. 1 spilled 36,000 sec.-ft.; San Gabriel No. 2 spilled 21,400 sec.-ft., and Santa Anita Dam, a variable-radius arch dam 230 ft. high with a spillway designed for 2,400 sec.-ft., passed 3,000 sec.-ft. for several hours, while water flowed 6 in. deep over the full length of the parapet.

The district's operating policy under these severe flood conditions was to keep outflow below inflow, the difference depending on rainfall in the mountains and storm predictions. Where one dam is above another having spillway gates, peak flow can be damped by gradual raising of the lower reservoir level; for example, with gates set to spill 25,000 sec.-ft. over Morris Dam, when inflows from the dam above tend to increase this discharge the gates can be partly closed to raise the overpour level without changing discharge quantity, thus providing 10,000 acre-ft. of added storage.

Channel improvements damaged

Verdugo Wash, carried in an open concrete channel through Glendale, a channel recently widened to take expected floodflow discharge, was not taxed to capacity. Heavy damage was done to the channel improvements, mainly in the upper reaches. The lower sections of the Los Angeles and San Gabriel river channels functioned well, and suffered little damage to levees.

All the debris dams functioned well. Seven debris basins in the Montrose-La Crescenta area above Glendale caught a total of 600,000 cu. yd. of debris. Fortunately their capacity was just sufficient to store the debris brought down. The basins will have to be excavated before they will again be effective, but in this storm they saved the area below from something worse than flood waters, namely debris floods such as those of New Year's Day 1934 (*ENR* Jan. 4, 1934, p. 22). Two similar canyons nearby, without check dams because values below are less, discharged debris flows resembling those of 1934. After filling, the debris basins formed delta cones on a slope of 7 to 10 per cent, whereas only 5 per cent was assumed in design.

Heavy damage occurred where the Los Angeles and Big Tujunga rivers join. Here overflows flooded highly developed residence property in the North Hollywood area. All bridges for six or seven miles along the Big Tujunga above the junction went out.

The city engineer of Los Angeles reports that modern permanent bridges are undamaged except in their approaches. Two old concrete bridges were lost; Lankershim Boulevard crossing of Los Angeles River, thirty years old and due for replacement, and the two 65-ft. arches over the Arroyo Seco at 43rd Ave., built in 1914. Of the city's less important bridges, mostly old wooden structures, 15 were lost or seriously damaged and 13 more had approaches destroyed.

Estimated replacement costs for damaged city structures total about $3,000,000, including bridges $908,000; sewers $350,000; storm drains $600,000; street and highway repairs, largely near channels $650,000; general clean-up expense $500,000.

Aqueduct not damaged

No further movement of Elysian Park slide occurred. No damage to the Colorado River Aqueduct was reported nor anything serious along the Owens Valley Aqueduct. However, considerable exposure of distribution mains occurred in San Fernando Valley.

The Los Angeles County Road Department estimates that replacement of damaged bridges will cost $700,000 and repair of county road damage $1,250,000. State highway officials estimate the cost of repairing bridges and damaged roads on the state system in Ventura, Los Angeles and Orange Counties at $1,500,000.

At Long Beach, while three concrete

bridges over the Los Angeles River stood safely ten old wooden bridges went out or were closed to traffic. One wooden footbridge collapsed suddenly on March 2 with loss of six or seven spectators by drowning.

Santa Ana runs wild

Orange County was hard hit by the flood. The Santa Ana river after leaving its mountain canyon had no further restraint, and spread out over its banks, flooding a wide area. At the ocean it cut new discharge outlets: here a width of three miles was submerged.

City and county structures suffered damage estimated at 1½ million dollars, including several bridges destroyed and others damaged. Part of a joint outfall sewer collapsed. The Santa Fe railway lost one bridge and the Southern Pacific two, and the coast highway lost several. Private property damage will reach 3½ millions, in the opinion of county assessors. Damage to public utilities, highways and railroads may easily reach 1¼ millions, bringing the total loss in Orange County above 6 millions.

Red Cross officials reported that they were temporarily sheltering and feeding 3,000 persons driven from their homes in Orange County, chiefly in Anaheim and Fullerton; 4,000 in Los Angeles County; 1,000 in Riverside County and 500 each in Ventura and San Bernardino Counties.

Red Cross figures on known dead by counties on March 5 were: Ventura 3, Riverside 15, San Bernardino 17, Los Angeles 32 and Orange 52. Homes damaged or destroyed in Orange County are placed at 2,100, in Los Angeles County 1,755.

County flood-control officials added a dramatic touch to operating practice when a Navy amphibian plane was obtained to aid 150 persons, some of them district employees, marooned without sufficient food at San Gabriel Dam No. 1. The plane made two trips on March 5, leaving 600 pounds of food each trip; later trips were planned as needed. Although the canyon is deep and tortuous, the full reservoir affords sufficient runway by landing headed inland and taking off down canyon.

Rail and road damage

Rail lines suffered heavily from undermined track, loss of trestles, bridge approaches and a few bridges. The Southern Pacific lost numerous trestles and two 100-ft. steel girder spans in the Los Angeles River crossing. Union Pacific and Santa Fe systems also had main lines inoperative for several days.

All main highways into Los Angeles were out of service for a short time at least, and some routes will not be fully opened until heavy slides are removed. However, highways were passable before rail routes reopened, and fleets of motor buses removed passengers from marooned trains.

Additional protection necessary

Early surveys of the flood conditions and damage are held to indicate the need for extensive additional protective measures. Unrestricted subdivision and building development in areas subject to possible overflow is recognized as a big factor in the losses sustained, and it is believed that strict regulatory measures to prevent or limit such development will have to be instituted. A planning board or other body of suitable competence and authority should administer such regulation and should maintain warning markers to indicate probable overflow areas, in the opinion of well-informed engineers. The matter is believed particularly urgent because some overflow areas are potentially much larger than the authorities have heretofore realized; the danger from the extreme floods that may be expected periodically is seen to be very serious, but measures to protect such property would be of prohibitive cost.

Present conditions also point strongly to the need for detention reservoirs on the coastal plain itself, over and above dependence on the reservoirs in the mountain valleys. The capacity of the mountain reservoirs that has been lost because of debris accumulation is unknown, but is probably large. Further, mountain storage is of no value against rainfall on the areas below.

Rainfall and Damage

Washington, D. C., March 7—Rainfall in the Los Angeles area during the four-day storm of last week was the heaviest recorded for any comparable period during the 61-year life of the station. One 12-hr. period showed 4½ in. of precipitation, and streamflow approached the theoretical 50-year maximum. Ballona Creek carried an estimated flow of 22,000 sec.-ft. The peak flow of the Los Angeles River is placed at 80,000 sec.-ft., an all-time record.

Incomplete reports to the Chief of Engineers on the functioning of the supplementary flood-control works that have been under construction there during the last two years by WPA forces indicate that the completed sections worked successfully and came through the flood without material damage. One section of stone-paved sand levee failed by scour of high-velocity flow, the estimated velocity being 30 ft. per sec. Had the projected retarding basins above this point been completed, however, flow here would have been much lower. Debris basins and improved channels at La Crescenta, Ballona Creek, Kenter Canyon, Rubio Wash and Sycamore Wash operated well and came through the flood undamaged.

Work still under construction suffered much damage, but details are not yet at hand. Alhambra Wash and Verdugo Wash in particular suffered damage, in addition to the Los Angeles River levee just mentioned.

Ten major bridges and many smaller structures were destroyed, but none of these was part of the WPA project. While three bridges across the Los Angeles River in a section of improved channel completed last year were lost, these had not been constructed as part of the project and constituted constrictions in the channel.

A bill has been introduced in Congress directing the Army Engineers to reconstruct all destroyed bridges over the controlled channels. A second bill would direct a survey of streams in Los Angeles and Ventura Counties for flood-control purposes.

Rehabilitation Loans

President Roosevelt on March 8 instructed the Reconstruction Finance Corporation to grant rehabilitation loans in the Southern California flood area. He acted in response to a plea by Governor Frank Merriam, who estimated public and private property damage at more than $52,000,000.

• • •

Hackett Named President Of Thompson-Starrett

Horatio B. Hackett who resigned last September from his position as assistant administrator of the PWA (*ENR*, Sept. 9, 1937, p. 441), has become president of the New York contracting firm of Thompson-Starrett Co., Inc., effective March 1.

Since leaving PWA, Colonel Hackett has been vice-president of the Chicago contracting firm of Coath & Goss as well as the Chicago Venetian Blind Co.

Colonel Hackett was in Washington for three and a half years, first as director of housing of PWA and later as assistant administrator. Before his service with the government, he was a partner in the firm of Holabird & Root, Chicago architects.

At the same time that Col. Hackett was appointed, L. J. Fischer, who has been connected with Thompson-Starrett for the past 40 years, was elected chairman of the board. During the past few years Mr. Fischer has been devoting a large part of his time to construction of the Grand Coulee Dam, on which the company is a contractor.

TVA Directors' Feud Stirs Congress

Charges by chairman and the board members make investigation by Congress certain

Publication of charges and counter-charges by Arthur E. Morgan, chairman of the Tennessee Valley Authority, and directors Harcourt A. Morgan and David E. Lilienthal within the past week brought the long-suppressed feud of the TVA directors into the open and resulted in action that carries assurance of a full investigation by Congress. Early in the year, when evidence of fundamental disagreements between the chairman and the two other directors of the TVA was brought to light by testimony of Arthur Morgan in condemnation proceedings involving a marble deposit under the Norris reservoir controlled by Senator George L. Berry and others. Senator Norris asked Congress to authorize an investigation of the TVA by the Federal Trade Commission (*ENR*, Jan. 6, p. 4). That proposal was strongly opposed in Congress as attempt to "drag a red herring across the trail of a smouldering scandal." However, no action was taken by Congress.

Berry claims rejected

On March 1 the federal commission that had been conducting the condemnation of the Berry marble deposit announced that it had found the deposit to be without commercial value. This brought from Chairman Morgan a full statement concerning the Berry claims in which he described how his efforts to prevent the other members of the TVA board from paying anything for the deposit now declared worthless had met with continued opposition from them. The Berry marble case, he said, "represents the kind of difficulty with which I have been faced in the effort to maintain good standards of public service." This, he added, is no family quarrel as the public has been led to believe. "The real difficulty has been in the efforts to secure honesty, openness, decency and fairness in government."

Immediately following the publication of this statement by Arthur Morgan the President made public a statement made to him by H. A. Morgan and David Lilienthal on Jan. 18. The latter statement charged Arthur Morgan with having progressively obstructed the work of the board, with having impugned the integrity of his associates, with having refused to carry out explicit action of the board and with having participated in the preparation of a power-pooling report which proposed violations of the TVA Act. Finally the statement charged Chairman Morgan with having adopted a "rule or ruin" policy and intimated that the other directors thought he should resign.

Morgan counter charges

Publication of this statement by the President was followed immediately by publication by Chairman Morgan of a letter he had written to Representative Maverick setting forth more details of the fundamental differences between the chairman and the other board members. In this he outlined how the two directors had consistently worked together to carry through policies to which he was opposed and how they even blocked his efforts to record his objections on the minutes of the board meetings; how they had announced a TVA power policy without any formal action on such a policy by the board and how power contracts had been kept from his scrutiny in advance of board meetings at which formal action by the board was required.

An unbiased examination of the electrical program of the TVA, said Chairman Morgan "will disclose disorder, waste, confusion and lack of planning to a startling degree. . . . The Berry marble claim, in my opinion, was an effort at a deliberate, barefaced steal. The other two directors had the same evidence that I did."

In conclusion Dr. Morgan said, "I am nearly 60 years old and many years ago deliberately gave up expectation of a public carrer. I did not seek my present position in any way. It would be pleasanter to resign and do some of the many things I am anxious to get at. Yet, to surrender the chance to make some contribution to decency and effectiveness of the government does not seem to be the right course."

• • •

Company and TVA Study Knoxville Purchase

Representatives of the Tennessee Valley Authority, the Tennessee Public Service Co., and the city of Knoxville met in Knoxville Feb. 28 for the purpose of negotiating the sale of properties of the Tennessee Public Service Co.

A contract was arranged in July, 1934, between the company and TVA providing for the sale of electrical facilities of the company to the TVA and the city of Knoxville. (*ENR*, Feb. 10, 1938, p. 196). Litigation, however, prevented consummation of that transaction.

Macdonald Answers Hurley Charges

Connecticut highway commissioner replies to public works commissioner's report

Highway commissioner Macdonald of Connecticut on March 1 submitted to Governor Cross a 150-page report replying to charges against the administration of the highway department made in a report by public works commissioner Hurley (*ENR*, Feb. 3. 1938, p. 162).

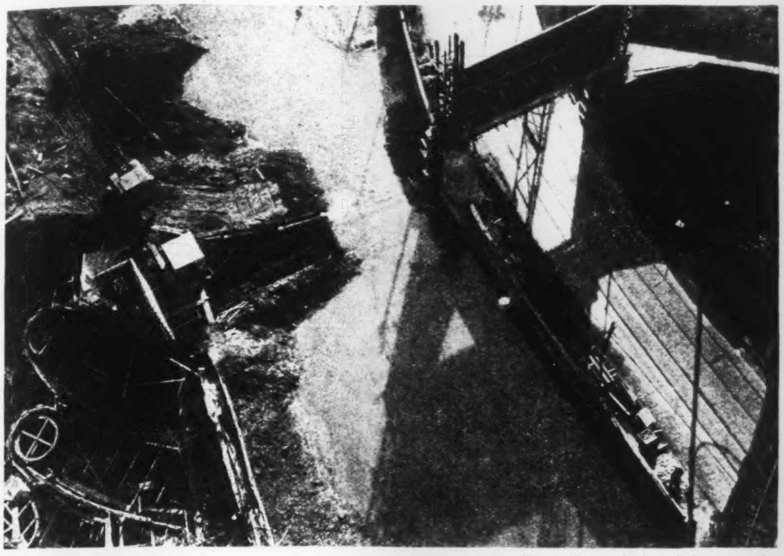
The Hurley report on the condition and operation of the state highway department, submitted last January, recommends the complete reorganization of the department. It bases the recommendation on charges of faulty planning, irregular activities, unsound engineering policies, inadequate inspection facilities, and a general lack of coordination. In particular the report charges favoring of certain contractors, relaxation of specifications, and permitting extra payments above contract prices.

Originally the Hurley survey was to determine whether the state could safely continue to build roads without resorting to bond issues and increased taxes, but the report went much beyond this subject.

The dispute between the two departments arises out of the 1937 Department of Public Works Act. Since this law was passed, the highway department has been operating under conditions of considerable uncertainty. After a decision of the attorney-general, highway construction and maintenance were handed over to the commissioner of public works, who had jurisdiction over this work for seven months. Then some of the control was returned to Commissioner Macdonald by agreement, and last month a decision of the Supreme Court of Errors returned the control of highway work to the state highway department (*ENR* Feb. 10. 1938, p. 195).

Macdonald attacks report

Discussing the general nature of the Hurley report, Mr. Macdonald asserts that the public works commissioner had no legal authority to carry on an investigation of the highway department, that he "chose to pursue his inquiries by under-cover methods" and that he did not seek the cooperation of the highway department. The commissioner attributes to these facts any lack of cooperation which Mr. Hurley found. He further asserts that the report "is not a temperate or a careful statement of facts . . . but is a series



Westinghouse Technical Press Service

PROTECTING A CREEK AGAINST FLOODS

Turtle creek flood control project, designed to protect the East Pittsburgh works of the Westinghouse Electric & Mfg. Co. from a recurrence of the St. Patrick's Day flood of 1936, is here shown under construction at the lower end of the creek.

The flood hazard arises from backed up waters of the Monongahela River, a mile distant, rather than from the creek itself.

The project consists of a dam across the creek to hold back the rising Monongahela and a high capacity low head pumping station which will lift the entire flow over the dam.

The pumping station, major item in the project, will consist of three propeller type pumps, each capable of lifting 1,122,000 g.p.m. against a head of 4 ft. or 897,000 g.p.m. against an 11-ft. head. The four-bladed propellers are 12-ft. in diameter and are mounted on 42-in. hubs. Each pump is driven by a 5,000 hp. synchronous motor operating at 112.5 r.p.m.

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of generalizations, misrepresentations, exaggerations, denunciations and sweeping allegations."

The major portion of the bulky report is devoted to a detailed analysis of the Hurley criticism, including a number of photographs which, when compared with photographs in the Hurley report, indicate, according to Mr. Macdonald, that Mr. Hurley used trick photography to produce an appearance of bad alignment of the Merritt Parkway.

Mr. Macdonald denies in his report that condemnation is to be preferred to direct purchase as a means of obtaining rights-of-way. He states that "most awards for damages as reassessed by the court have greatly exceeded the original awards made by the department on the basis of independent and authoritative appraisals."

Recommended changes

Commissioner Hurley's report had proposed that the highway department be reorganized into districts and had suggested a greatly reduced budget for the department.

Commissioner Macdonald asserts that Mr. Hurley's budget neglects to provide for a large number of functions which the highway department has to carry out and says that a state as small as Connecticut is not well adapted to the district type of highway organization. He points out that many highway districts in other states are larger than the entire state of Connecticut.

• • •

Sharp Pencil Bidding

Three bids with a spread of only $13 on a $108,000 job were submitted on Feb. 24 for the power, control and lighting system of Lock and Dam 13 on the Mississippi River. A low figure of $107,800 was submitted by the Dearborn Electrical Const. Co., Chicago; second low was that of the Sterling Electrical Co., Minneapolis, $107,811; third was the $107,813 proposal of the Federal Engineering & Const. Co., Kansas City. Bidding was on 29 lump sum items. Despite the closeness of the totals, there was a wide spread on individual items in the three low bids, one item running from $252 to $1,130.

Ohio Engineer Fired By Governor

Bituminous engineer dropped from highway department after testifying on corrupt practices

Harry A. Sparks, bituminous engineer in the Ohio state highway department, was ordered dropped from the state payroll on March 3 by Governor Martin L. Davey. The governor ordered state highway director Jaster to abolish the position which Mr. Sparks held, which would make redress through the Civil Service Commission impossible.

Mr. Sparks' dismissal was foreshadowed in statements made by Governor Davey in an address before the lower house of the Ohio General Assembly Feb. 28, when the Governor appeared before that body to urge it not to vote for an extension of the session which would give the Senate's special investigating committee more time to conduct its probe of the state government.

During the weeks preceding the Governor's speech, a number of sensational charges had been made concerning alleged irregularities in the highway department. Mr. Sparks was one of the principal witnesses subpoenaed before the Senate committee and contributed much of the testimony. In his testimony he stated that estimates were padded in highway contracts so that a "ring" of bituminous road material men could make exorbitant profits. He said that the state paid $14 per cubic yard for bituminous material which the federal government buys at $6.56.

Mr. Sparks also charged that the state had wasted $2,000,000 on bituminous roads in the past two years and said that when certain favored contractors were faced with competition in bidding on road projects, the projects were cancelled until the competitors got tired of bidding and let the "ring" take the contract.

In discussing the testimony, Governor Davey described Mr. Sparks to the state legislature as being "inefficient, disloyal and contemptible," and said that he intended to "fire" him.

Mr. Sparks is a graduate in civil engineering of Miami University. He has served as construction engineer on several electric traction lines in Ohio and as city engineer of Xenia, O., and Lansing, Mich. He later engaged in the highway contracting business and then practiced as a consultant.

His first connection with the state highway department was made in 1929 when he became field inspector of asphalt highway construction.

J. A. L. Waddell Dies At 84

Noted bridge builder was originator of modern type of vertical lift bridge

John Alexander Low Waddell, for two generations one of the best-known of American bridge engineers, died at New York on March 3 at the age of 84 years, following a stroke last November.

Dr. Waddell was born at Port Hope, Ontario, in 1854. After a trip to China at 16 he entered Rensselaer Polytechnic Institute in 1871, and graduated in 1875. The next six years he spent partly in railway work and partly in teaching at Rensselaer, following which, in 1881, he became chief engineer for Raymond & Campbell, bridge builders, of Council Bluffs, Iowa. In 1882 he went to Japan, where for four years he was professor of civil engineering in the Imperial University at Tokyo.

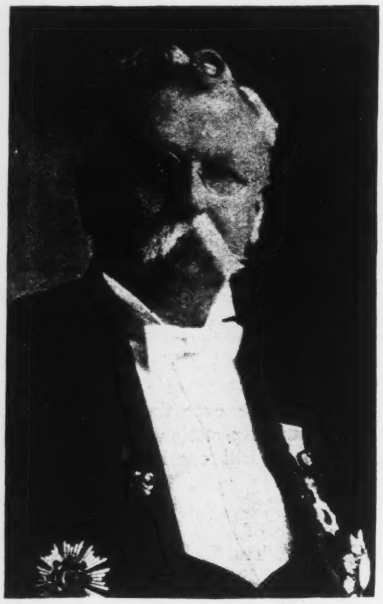
Shortly after his return to the United States Dr. Waddell opened a consulting office for the practice of bridge engineering in Kansas City, Mo., where he continued until 1920, when he moved his office to New York City. He practiced under his own name until 1899, then as Waddell & Hedrick (1899-1907) and Waddell & Harrington (1907-1917). In 1917 he entered into partnership with his son, N. Everett Waddell, since deceased. From 1920 he practiced engineering under his own name until 1927, at which time he took into partnership his principal assistant engineer, Shortridge Hardesty, under the present firm name of Waddell & Hardesty.

Beginning with 1887 Dr. Waddell devoted his entire professional career to the design and construction of bridges. Among other structures his work has included the Red Rock cantilever bridge over the Colorado River at the Needles, the Northwestern Elevated R.R. and the Union Loop Elevated R.R. of Chicago, the Mississippi River Bridge at Cairo, Ill., the Cooper River Bridge at Charleston, S. C., the Goethals Bridge at New York, and the suspension bridge over the Maumee River in Toledo, Ohio.

Dr. Waddell was the originator of the modern vertical lift bridge, his first structure of this type being the Halstead Street Bridge in Chicago, which was built in 1895. His other important structures of this kind include railroad crossings of the Hackensack and Passaic rivers, Newark Bay, and the Chicago and Calumet rivers, the Suisun Bay Bridge and the double-deck bridge over the Willamette River at Portland, Ore.

Dr. Waddell was twice called to China by the Chinese government, in 1921 as the American member of an International Commission for reviewing the designs submitted for a proposed bridge over the Yellow River, and in 1929 as a consulting engineer to the Ministry of Railways.

Dr. Waddell contributed liberally to the literature of bridge engineering.



J. A. L. WADDELL

His principal books were "The Designing of Ordinary Iron Railway Bridges" (1884), "De Pontibus" (1898), "Bridge Engineering" (1916), and "Economics of Bridgework" (1921). Among other achievements in this line he elicited and edited in 1933 a book to guide students in the choice of engineering as a profession, entitled "Vocational Guidance in Engineering Lines," which was prepared with the assistance of fifty of the leading engineers of the United States.

• • •

Muskingum Bond Issue Is Enjoined

A permanent injunction has been granted by the Fifth Ohio District Court of Appeals preventing the Muskingum Conservancy District from issuing $1,500,000 in bonds. The court held that the district lacks authority to levy general taxes on property not directly benefited by the district's flood control project.

Counsel for the district had argued that the principal purpose of the bond issue was the development of a recreation and conservation program in the Muskingum area.

First Incinerator At Detroit

First of four incinerators is put into operation as reduction contract ends

Operation began Feb. 25 in the first of four combined garbage and rubbish incinerator plants planned for the city of Detroit. The incinerators will replace the former method of disposing of the city's garbage, by which it was shipped out of the city to the Detroit Reduction Co.'s plant. The contract with the reduction company terminated Feb. 27.

The Northwestern plant, with a capacity of 450 tons per day, is the largest of four plants provided for in a $1,400,000 WPA project. The Central plant, with a capacity of 350 tons, will go into operation early in March, and the Eastern and Western plants, with 350 and 250 tons capacity, respectively, are now at the foundation stage of construction.

In preparation for the shift to incineration, the city council has enacted an ordinance requiring that all garbage be drained and wrapped and that combustible waste be separated from the non-combustible, which will be collected separately.

The contract for construction of the four plants was awarded in January of last year to the Underpinning & Foundation Co. Inc. and Decarie Incinerator Corp. on a bid of $1,331,000. A PWA grant provided 45 per cent of the cost and the remainder was financed by Wayne County by a bond issue sold to PWA. County financing was necessary because the city had reached its debt limit. A liquidating service charge paid by the city will enable the county to pay operating charges and retire the bonds.

The new method of disposal is expected to save several hundred thousand dollars over the payment previously made to the Detroit Reduction Co., which received $2.25 per ton for garbage handled.

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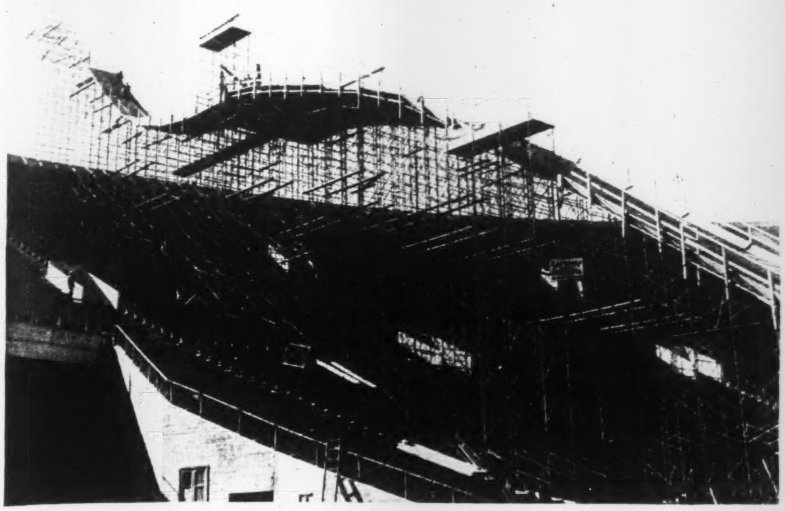
Thousand Islands Bridge

In connection with an aerial perspective of the Thousand Islands Bridge published in our Feb. 24 issue, p. 282, the statement concerning the design of the bridge was incomplete and inaccurate. The bridge was designed by Robinson & Steinman, consulting engineers, in charge of design and construction of the entire enterprise. Monsarrat & Pratley, consulting engineers, Montreal, Canada, are associate engineers.



Acme Photo

ARTIFICIAL MOUNTAIN FOR SKIING

SAID to be the largest man-made ski slide ever built, this structure in the Los Angeles Coliseum is for the Southern California open jumping meet, and rises 60 ft. above the tip of the Coliseum bowl. About 150 tons of ice will be put through a snow machine to form the blanket for the slide.

Utility Company Contracts For Boulder Power

The Citizens Utility Co. of Minneapolis which operates the power system serving Kingman County, Ariz., has completed a contract by which it may buy 50,000 kw.-hr. annually from the Boulder Dam power plant until Dec. 31, 1954. The power being sold is firm power originally contracted for by the Metropolitan Water District of Southern California.

A similar contract was completed last month by which the Needles Gas & Electric Co. of San Francisco which serves Needles, Calif., obtained the right to buy 20,000 000 kw.-hr. annually of the water district's energy. This contract also extends to Dec. 31, 1954. Both utilities are paying the regular price for firm energy at Boulder Dam, 1.63 mills per kw.-hr.

The Metropolitan Water District intends to use 36 per cent of the Boulder Dam power to pump water through the Colorado Aqueduct. Since Boulder Dam was completed more than two years ahead of schedule, while the aqueduct program could not be advanced ahead of schedule, the district has been unable to make use of the power. Under a provision in the contract with the water district, the Secretary of the Interior is empowered to dispose of unused energy, after first offering it to the city of Los Angeles and other major purchasers who hold options on the unused power.

The Citizens Utility Co. plans to build a transmission line to the power plant, and the government will install transformers and certain other equipment which the company will pay for at 4 per cent interest in ten years. The company also agrees to pay a proportionate share of the cost of the turbines and generators installed to serve the Metropolitan Water District. These charges will be worked out between the company and the district.

• • •

Quarry Labor Agreements Call for Dust Control

Labor agreements now being consummated between operators of granite quarries in Vermont and surrounding states and the Quarry Workers International Union of North America provides for installation of rock drill dust control equipment. The agreements specify that the dust control equipment must be installed on or before June 1, 1938, and that the equipment must be of a type approved by the Industrial Board of the state of New York under rules of test procedure adopted by the New York industrial commissioner in May, 1937, as provided by the state's Silicosis Act. Vacuum exhaust control devices, already approved by New York state, are now being placed in granite quarries in the Barre area. John C. Lawson, of Barre, secretary of the quarry workers union, reports full cooperation of the employers in this move to combat the silicosis hazard.

Reclamation Charges Are Postponed

Ickes accepts committee's recommendation that part of reclamation repayments be deferred

Seven recommendations extending the time for 1937 payments due on certain reclamation projects have been approved by Secretary of the Interior Ickes. These bring to a close one part of the work assigned by Congress to a special repayment commission appointed last November by Sec. Ickes.

The commission is composed of Dr. Charles A. Lory, president of Colorado State College, William R. Wallace, chairman of the Utah Water Storage Commission, and George T. Cochran, former state water superintendent of Oregon. It was ordered by Congress to recommend what if any part of the federal reclamation payments due for 1937 should be postponed, and to recommend to Congress a new repayment method that will be more just and equitable than the old system of granting a blanket moratorium in times of economic stress. The latter task is still to be completed.

In discharging the first part of its duty the Commission made the following recommendations, which have been approved and are now effective:

Belle Fourche project, S.D.—That 70 per cent of the $55,000 construction repayment for 1937 be postponed until the fifth and sixth year after the completion of the repayment contract.

Newlands project, Nevada—That 50 per cent of $32.485.14 due for drainage construction be postponed, to be paid in 14 equal semi-annual installments beginning Jan. 1, 1947, and that 50 per cent of $38.446.48 due for project construction be postponed, to be paid in one installment in 1966.

Uncompahgre project, Colo.—That 50 per cent of $90.327.17 construction repayments be postponed, to be repaid in the sixth year after the completion of the present contract.

Gravity division, Minidoka project, Idaho—That all construction repayments in excess of $2.50 per acre be deferred. This will affect principally late entrants who have poorer lands.

Gooding division, Minidoka project, Idaho—That 50 per cent of $167.684.82 due be postponed until 1957.

Okanogan project, Wash.—That 50 per cent of $5.000 due be postponed until Dec. 1, 1965.

Yakima project, Wash.—As regards Warren Act lands in the Sunnyside Valley, 50 per cent of $7.500 now due is postponed until six years after the last installment under existing contracts.

WASHINGTON HIGHLIGHTS

Special reports by Paul Wooton

Resumption of large scale government spending this summer is expected by many Washington observers. Those who have been expecting the administration to make some dramatic confidence-building gesture are about ready to concede that they guessed wrong. The time before the elections is growing too short, for the effect of such policy to bring about recovery enough, they think. It is safe to assume that regular appropriations becoming available July 1 will be expended as largely as possible during the early months of the fiscal year.

President Roosevelt on March 4 signed a bill extending the loaning power of the Disaster Loan Corporation to cover catastrophes occurring during the current year. The corporation operates with funds provided by the Reconstruction Finance Corporation in areas needing rehabilitation financing as the result of floods, tornados and other disasters.

A new draft of the regional authorities bill, commonly called the seven TVAs bill, has been submitted to the Rivers and Harbors Committee by administration authorities. While the administration has made concessions in each succeeding draft presented it is understood that even the latest proposal is unacceptable to a majority of the committee members. Provisions for local contributions, made in the administration drafts, are not entirely satisfactory to some of the members of the committee.

Prospects are not bright for the Parsons bill, providing for 5,000 second feet of diversion from Lake Michigan at Chicago. The House rivers and harbors committee has laid the bill aside ostensibly to allow members to study the mass of testimony that has been taken. This action was taken over the protests of the proponents of the bill, who realize that such legislation will become increasingly difficult to secure as work progresses on the Illinois river dams.

Members of the conference committee to which the stream pollution bill (*ENR*, Aug. 19, 1937, p. 293) was referred after it had passed both houses of Congress admit that after seven months there is little prospect that they will report. The bill dies with the Congress at the end of this session if it is not possible to reach a compromise. A majority of the conferees are in sympathy with objections raised by state and municipal authorities, who say the bill would put the federal government in a position to require costly improvements which might be in conflict with the views of local officials and the state of the bond market.

The super-highway bill sponsored by Senator Bulkley, which would have provided for an $8,000,000 program to construct express highways across the country, was apparently removed from any possibility of immediate action when it was transferred by a Senate vote from the Banking and Currency Committee to the Senate Committee on Post Offices and Post Roads. Senator Bulkley was chairman of a subcommittee considering the bill when it was in the Banking and Currency Committee.

Cement Price Fixing Charged in Texas

Attorney General McCraw of Texas filed suit March 7 against six large cement manufacturing companies charging violation of the Texas anti-trust laws. The attorney general charged before the district court at Austin that the companies had entered into a price fixing agreement early in 1929 at the time of the formation of the Cement Institute, with headquarters in Chicago. He charged that "a code of ethics" and a "compendium of established terms and marketing methods" adopted by the cement manufacturers had led to identical bidding practices.

The attorney general seeks to have the charters of the companies cancelled and also seeks penalties of from $50 to $1,500 a day against each of the defendants for the period from Jan. 3, 1929 to March 7, 1938. The penalties might total as much as $30,000,000.

The companies named as defendants in the suit are: Lone Star Cement Corp., New York City and Dallas; Southwestern Portland Cement Co., Los Angeles and El Paso; Trinity Portland Cement Co., Chicago and Dallas; Universal Atlas Cement Co., a subsidiary of U. S. Steel Corp., Chicago and Dallas; Longhorn Portland Cement Co., San Antonia, and San Antonio Portland Cement Co., San Antonio.

St. Louis Civil Engineers Celebrate Anniversary

Nearly two hundred members and guests of the St. Louis Section of the Am. Soc. C. E. assembled at dinner on the evening of February 28 to celebrate the Golden Anniversary of the section. Immediately following dinner a cast, composed of members of the section, enacted in dramatic form a reconstruction of the first meeting of the St. Louis members of the Am. Soc. C. E., as it occurred on Feb. 29, 1888.

Those who thus organized the section and were represented in the presentation were: Henry Flad, then president, Board of Public Improvements; M. L. Holman, water commissioner; S. Bent Russell, principal assistant engineer, Water Department; Thos. Whitman, former water commissioner, and brother of Walt Whitman; Carl Gayler, bridge engineer; J. B. Johnson, professor of civil engineering, Washington University; Robert Moore and R. E. McMath, consulting engineers.

Guests of honor were Baxter Brown, past-president of the section, and past-director Am. Soc. C. E.; R. C. Gowdy, vice-president, and George T. Seabury, secretary, Am. Soc. C. E.; and representatives of other branches of the profession. A message of regret was read from S. Bent Russell, past-president and only living member of the group which organized the section.

An address "How Long Is Fifty Years?" was delivered by Col. Willard Chevalier, vice-president, McGraw-Hill Publishing Co. and past director, Am. Soc. C. E. Toastmaster was Wm. C. E. Becker, president of the section.

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Local Flood Works Urged On Connecticut River

Authorization of additional local protective works on the Connecticut River was recommended March 5 in a report to Congress by Major-General Julian L. Schley, Chief of Engineers. "The great flood of March, 1936, has conclusively demonstrated," according to the report, "that the flood problem on the Connecticut River is much more serious than previously considered.

Recommended are dikes to protect the cities of Hartford and East Hartford, Conn., and Springfield, West Springfield, Chicopee, Holyoke, and Northampton, Mass. Additional reservoir authorizations, however, are held to be unnecessary at this time. Six Connecticut valley reservoirs are already authorized, but construction is delayed by the pending dispute between the states and the federal government as to the ownership of power.

News continued on page 381

THE CURRENT TREND

This week's news of contract volume and construction cost factors

Gain in Urban Housing

Dwelling units in 1937 6 per cent above 1936, highest year since 1930.

New Family-Dwelling Units in Urban Areas per 1000 Urban Population
1936 and 1937

	1936	1937
United States	**2.88**	**3.06**
New England	**1.56**	**1.65**
Connecticut	2.00	2.39
Maine	1.54	1.69
Massachusetts	1.47	1.42
New Hampshire	1.31	1.50
Rhode Island	1.49	1.79
Vermont	1.41	1.33
Middle Atlantic	**2.63**	**2.90**
New Jersey	1.77	1.53
New York	3.82	4.29
Pennsylvania	1.13	1.39
E. North Central	**1.81**	**1.80**
Illinois	1.14	0.99
Indiana	1.09	1.83
Michigan	2.31	2.95
Ohio	2.06	1.80
Wisconsin	3.23	2.23
W. North Central	**2.12**	**2.14**
Iowa	1.79	1.79
W. North Central, Cont.		
Kansas	2.26	2.42
Minnesota	2.14	2.54
Missouri	1.99	1.86
Nebraska	2.50	2.22
North Dakota	2.78	2.05
South Dakota	3.58	3.12
South Atlantic	**4.82**	**4.73**
Delaware	3.16	1.70
Dist. Columbia	13.08	10.97
Florida	10.61	9.78
Georgia	2.65	2.99
Maryland	1.65	2.09
North Carolina	4.14	4.74
South Carolina	4.96	4.25
Virginia	2.58	2.68
West Virginia	2.84	3.46
E. South Central	**3.40**	**2.90**
Alabama	2.78	2.65
Kentucky	2.70	2.16
Mississippi	5.06	4.71
E. South Central, Cont.		
Tennessee	3.91	3.08
W. South Central	**4.35**	**4.84**
Arkansas	2.46	2.39
Louisiana	2.77	2.92
Oklahoma	3.93	4.41
Texas	5.34	6.04
Mountain	**3.89**	**4.43**
Arizona	3.72	4.31
Colorado	2.58	3.27
Idaho	7.09	5.93
Montana	3.95	3.79
Nevada	8.86	10.08
New Mexico	6.75	7.40
Utah	3.12	4.51
Wyoming	3.82	4.59
Pacific	**5.74**	**6.62**
California	6.85	7.77
Oregon	2.33	3.87
Washington	2.37	2.74

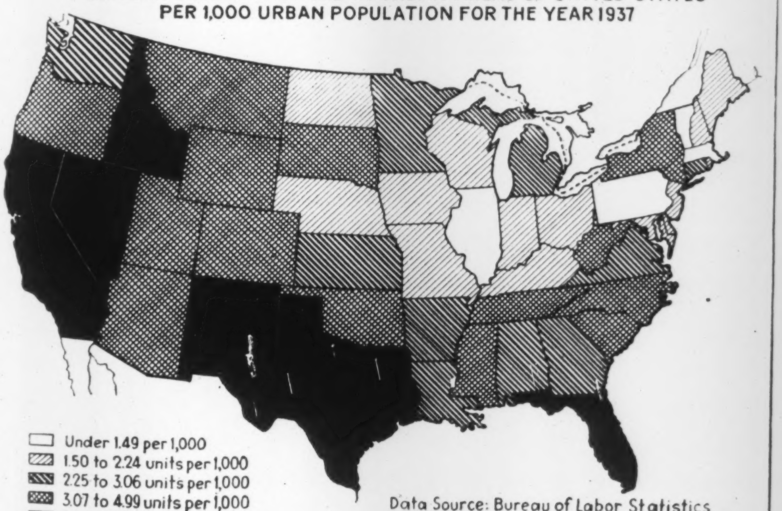
New urban dwelling units built in United States during 1937 provide for 211,265 families, an increase of 6 per cent or 12,292 units over those provided last year. This, the highest total since 1930 when 224,000 units were built, is only 28 per cent of the 752,000 units built in 1925, the peak year reported by the Bureau of Labor Statistics.

In analyzing the 1937 building permit reports of 1500 cities, representing 85 per cent of the entire urban population, dwelling units provided range from 0.99 units per 1000 urban population in Illinois to 10.97 per 1000 in the District of Columbia. Nevada follows District of Columbia with 10.08 units, Florida is next with 9.78, then California with 7.77, New Mexico with 7.40, and Texas with 6.04. The United States average is 3.06 units per 1000, compared with 2.88 a year ago. Sectionally, the three Pacific Coast states provide the greatest number of dwelling units per 1000 urban population, 6.62. This compares with 5.74 in 1936. New England states provide 1.65 units per 1000, compared with 1.56 last year.

Gains over 1936 in cities, by population groups, range from 3 per cent in cities of 500,000 and over, to 17 per cent in those of 10,000 to 20,000 population. In the 100,000 to 500,000 population group there is a 5 per cent drop from 1936.

One-family dwelling units in 1937 constitute 69 per cent of the total compared with 66 per cent in 1936; two-family units, 7 per cent, compared with 6 the previous year; and multi-family dwelling units 24 per cent, compared with 28 a year ago. Middle Atlantic states, in providing 31,347 units in multi-family dwellings, are the only section in which multi-family exceeds one-family dwellings. In New England, East and West North Central, East and West South Central, and Mountain states, more than 80 per cent of the structures are single-family dwellings.

DWELLING UNITS PROVIDED IN URBAN AREAS OF UNITED STATES PER 1,000 URBAN POPULATION FOR THE YEAR 1937

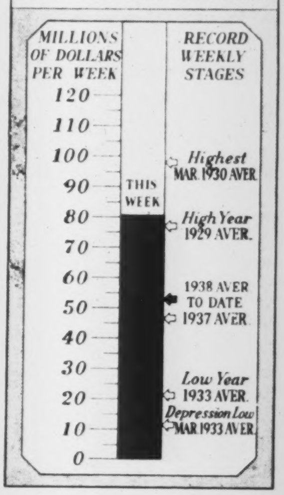


☐ Under 1.49 per 1,000
▨ 1.50 to 2.24 units per 1,000
▨ 2.25 to 3.06 units per 1,000
▩ 3.07 to 4.99 units per 1,000
■ Over 5.00 units per 1,000

Data Source: Bureau of Labor Statistics
Department of Labor

Computations by *ENR*

E.N.-R. Construction Volume



THIS WEEK'S FIGURES

(Thousands of Dollars)

CONTRACTS

	Week Ending Mar. 11 1937	Mar. 3 1938	Mar. 10 1938
Federal	$2,550	$4,965	$4,006
State & Municipal	17,097	19,076	18,616
Total public....	$19,647	$24,041	$22,622
Total private....	21,441	17,511*	57,902
TOTALS	$41,088	$41,552*	$80,524

Cumulative
1938 (10 weeks)............$521,743*
1937 (10 weeks)............$435,487

Note: Minimum size projects included are: Waterworks and waterways projects, $15,000; other public works $25,000; industrial buildings, $40,000; other buildings, $150,000. * *Revised.*

NEW PRODUCTIVE CAPITAL

	Cumulative 1937 10 Wk.	1938 10 Wk.
NON-FEDERAL	$289,069	$210,444
State & mun. bonds.	129,907	124,291†
Corporate securities..	117,986	86,280
PWA loans, grants...	41,169	—2,497†
RFC loans	7	2,370
FEDERAL	....	....
TOTAL CAPITAL ...	$289,069	$210,444

† Represents transfer from federal to private investment financing through sale by RFC of PWA bonds.

ENGINEERING NEWS-RECORD
Copyright 1938, by McGraw-Hill Publishing Company, Inc.
Volume 120. Number 10
Editor, F. E. SCHMITT · *Editorial Staff*, V. T. BOUGHTON
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Editorial Offices at 330 West 42nd Street, New York
MARCH 10, 1938

RUNNING STRONGLY counter to the wavering trends of commercial business, construction is going forward into the working season with a high and sustained volume of contract awards. The first ten weeks of 1938 show a total 21 per cent ahead of the previous year. January brought good figures, as already noted in this column, but they were even bettered in February, the highest of any February since 1929, and March is holding well up to this precedent. The fact that large housing enterprises are a significant item in the total suggests that the long overdue upswing in rehousing may be getting under way. But there is sustained activity in other major departments of construction. The recovery that began three years ago was one-sided, as construction lagged badly behind; the 1938 revival should be sounder because of the construction recovery that current figures presage.

Concrete Columns

CONCRETE COLUMN STRENGTH is placed on a sound experimental basis by the notable series of tests of columns under central and eccentric loading reported by Prof. F. E. Richart to the American Concrete Institute two weeks ago. The range of the tests and the consistency of the results makes them a reliable guide to design practice. Eccentric loading reduced the capacity of these columns closely in proportion to the amount indicated by combined-stress theory, which fact supplies as good a confirmation of reinforced-concrete calculation methods as could be asked. But the amount of the reduction was less than expected, the columns showing an excess resistance to bending just as do reinforced concrete beams. Prof. Richart concludes that columns subject to bending may be credited with a higher strength than those under direct compression. The conclusion is well supported so far as test facts go. Its application to working stresses, however, will involve consideration of the factors of safety applicable to different kinds of service. For example, a column that forms part of a rigid frame and carries bending only by reason of frame action is in less danger of collapse than one whose bending is the direct result of eccentric loading. In this sense the excess flexural strength revealed by the tests may not in all cases be equally available to the designer.

Expensive Pique

IT IS QUITE GENERALLY BELIEVED in Washington that the Interstate Commerce Commission delayed in acting on the railroads' application for rate increase, and perhaps also limited the tardily granted increase to a half-loaf, because the commission was peeved by the President's too obvious attempts to force it into a quick and favorable decision. Such pique is natural, but at a time like this yielding to it is an expensive luxury. Note for evidence in the current report of steel production the statement that the steel companies have inquiries concerning the price of 30,000 tons of steel rails, orders for which hinged on the I.C.C.'s findings. Similarly much work for the construction industry is in the balance. With spring approaching it was clear that the continued threat of railroad bankruptcy would indefinitely delay projects that could be put under construction this summer. Neither bureau pique nor narrow judgment should be permitted to perpetuate railroad insolvency and hold up needed work.

An Undesirable Change

NATIONAL FLOOD-CONTROL POLICY is to be changed by a new bill that is to be introduced into Congress with administrative endorsement. Under its provisions the federal government will pay all the cost of dams and reservoirs from which revenue can be obtained through the sale of power or irrigation water. Present requirements that local beneficiaries pay for land and flowage rights are to be eliminated. This proposed change comes as no surprise, since it was foreshadowed last spring when the Federal Power Commission blocked the start of work on flood-control projects in New England for fear that federal control over possible power development would not be complete. The new legislation is in line with the administration's policy of extending federal control to all possible power sites. Nevertheless it is an undesirable change. Requiring local beneficiaries to share in the cost of a project serves as a useful check on extravagance. It should not be eliminated.

Exploring Stress Distribution

A LEADING ELEMENT in the increasing power of structural engineering is the sureness and precision supplied by modern research, largely through definite measurement. Noteworthy examples of this type of research may be seen in the recently reported experimental studies of rigid frames and of concrete columns. In the field of steel one of the most powerful tools of economy and efficiency is the exploration of stress distribution by means of the strain-rosette method. It promises great influence on the details of future structural engineering practice—one example being the studies of stresses in riveted and welded knee frames carried on during the past year or more at the Bureau of Standards. As the method should be familiar in all its details to everyone who works in the structural field the clear statement given by Profs. Beggs and Timby in this issue is particularly welcome. It will have its most productive results if knowledge of the method will lead to applying this type of exploration to a wide range of structural forms and details. The distribution of stress in steelwork details still remains among the most obscure phases of structural knowledge.

Preventable Loss

FLOOD DESTRUCTION in the Southern California coastal area during last week's great storm may have been exaggerated by newspaper headlines, but it would be hard to overstate the extent of unnecessary loss of life and property that occurred. Though the floods matched or exceeded the worst that had been experienced in sixty years, yet in the main they swept the same valleys and channels that always have been flood territory. The people who built dwellings in the sweep of flood waters gambled against fate; the price of the gamble has been paid in scores of lives and millions of property lost.

It would be hard also to overstate the importance of this newest demonstration of the effectiveness of flood control. Flood peaks were reduced one-third to one-half by storage reservoirs; the sad fact that in spite of this protective action so much destruction was wrought should give impetus toward completing the protection works. Further encouragement is found in the fact that all well-built engineering structures survived the flood. Fifteen flood-control dams stood safely, many of them under all-time record flow. The bridges that were washed out were mainly old and inadequate structures, whose failure but emphasizes the value of adequate design and sound construction of the many more that stood unscathed.

Engineers have long known that the Los Angeles region presents one of the most critical flood problems in the whole United States. The public was slow to recognize this fact, however, and did little to force systematic protective work. Planning began a quarter-century ago, but confusion and controversy so delayed the work that only a small part of the needed protection has been constructed. It is fortunate that politics did not delay the final completion of the big San Gabriel Dam longer than it did; in this one flood the dam, completed only a few months ago, more than paid for its cost. Another year's delay in building it would have meant real disaster in the valley below.

In contemplating present losses and reflecting on what might have happened the people of Southern California may be roused to better realization of their situation and may come to have a respect for flood control that will be effective in preventing still greater losses at some future time. The lesson of this as of many an earlier flood is that floods can be controlled and flood losses prevented.

Adequate protection against future floods means vigorous prosecution of a complete flood-control program for the whole coastal plain and, simultaneous with it, firm exercise of public authority over channels and flow areas to exclude real-estate exploitation and community occupation. To provide a broader form of organization for the purpose than the inadequate county flood district system is the duty of the state. The actual control planning and construction is the task of the local community itself.

Time to Investigate

A PLAIN ISSUE is presented by the clash within the Tennessee Valley Authority that flared into the open last week. The issue raised by chairman A. E. Morgan is honesty and integrity in the discharge of a public trust. He confronts the country with the necessity for a searching investigation by Congress.

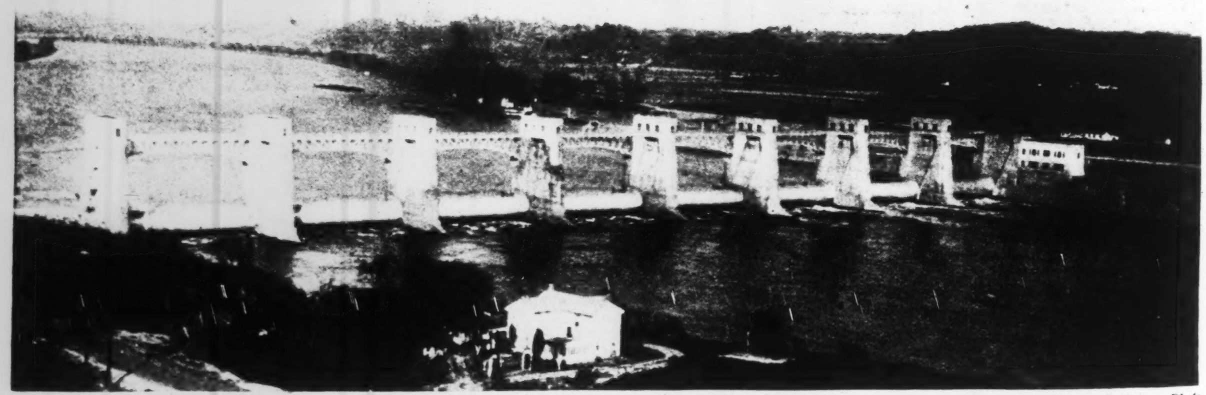
Back of the present issue lies a history of five years of dissension and intrigue within the TVA board that almost from the first destroyed the essentials of representative administration. A glaring example was the publication of power rates within a few months after the TVA had been established, a decision not reached by the board as a whole. From an early date arbitrary one-man procedure became the rule in the TVA, and the public's reliance on balanced and well-considered administration to safeguard its huge investment was betrayed. A proper investigation cannot avoid exploring this condition as well.

In establishing the TVA the nation entered into a great experiment, one that promised to show what can be accomplished in comprehensive river development and what methods of organization, financing and administration are adapted to such an undertaking. The experiment obviously called for wisdom and judgment, but even more vitally for unswerving rectitude and fairness. The present danger is that the TVA will fail for lack of these indispensable qualities. Indeed, in view of what happened in the matter of the Berry marble claims and in the light of Chairman Morgan's challenge, failure of the TVA may already be at hand.

PICTURES IN THE NEWS

Bureau of Reclamation Photos

WATER SPORTS AT GRAND COULEE. (Left) Safety vests in case of a drop into the swift and icy waters of Columbia River protect these men, who are operating "pinchers" for pulling piling from cofferdams. (Above) Nothing fancy about this diver, who was at the other end in pile-pulling operations.



Byron Bodimer Photo

GALLIPOLIS DAM IN OPERATION. Recently finished, this dam near Gallipolis, O., controls water levels in aid of navigation on a long section of the Ohio and lower reaches of the Kanawha. Its record roller gates are 125 ft. long and stop logs are aluminum trusses of the same length.

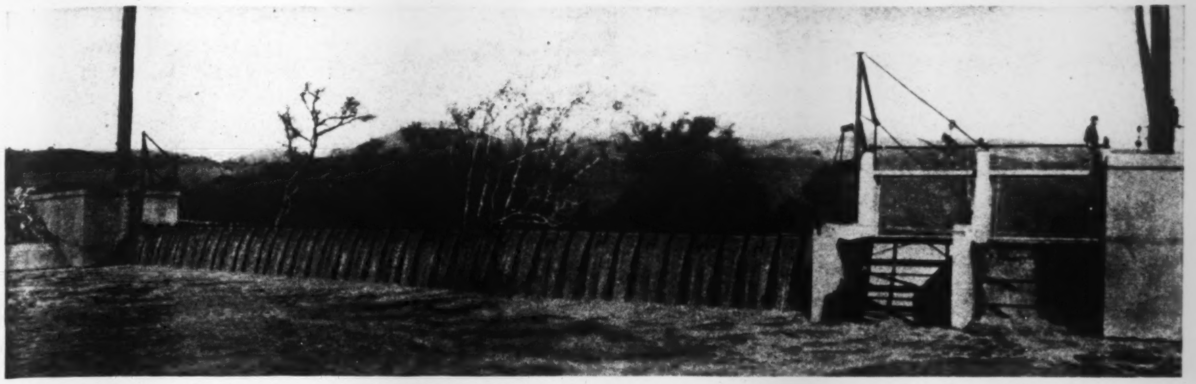


FIG. 1. THE PERCOLATING DAM ON COYOTE CREEK IS RAISED WHEN WATER IS CLEAR, LOWERED WHEN WATER IS SILTY.

Renewing Underground Water Supply

by F. H. Tibbetts

Chief Engineer, Santa Clara Valley Water Conservation District, San Francisco, Calif.

Data on operation of percolating areas is accumulated and applied successfully in renewing the underground supply of California's Santa Clara Valley

Underground water storage basins in the Santa Clara Valley, Calif., are being replenished successfully with the aid of percolating areas in which are spread flood waters that have been held temporarily in surface storage behind five dams built for this purpose. Characteristics of the underground basins are now fairly well understood, there has been ample justification for the construction program that was adopted by the conservation district in 1929 and since water spreading operations have been carried on valuable experience with percolating areas has been accumulated. Operating technique and results are outlined in the following.

A description of conditions obtaining in this district, underlying soil formations, and an outline of the general plan under which the storage dams were built, were published in *Engineering News-Record*, July 2, 1936, p. 10.

Conditions in the Santa Clara Valley are favorable to replenishment of underground storage because the 130,000 acres of highly developed agricultural land constituting the district is underlain by a mixture of materials eroded from the surrounding hills and heavily interlaced with water-bearing gravels of varying thickness. Well logs indicate that about 10 per cent of the valley fill which is tapped by wells consists of water-bearing gravel or sand. These water-bearing materials, or aquifers, appear to be buried stream-channels of much the same character as the gravel or sand layers of a few feet in thickness that occur in the bottom of the present active stream-channels.

The aquifers radiate from large alluvial cones of gravels and boulders on the present streams, they follow roughly the general direction of the present surface drainage and frequently inter-connect. The aquifer material, like that in beds of the present streams, is quite open and porous. The theory is that as the stream channels changed repeatedly in the process of building up the valley floor, the aquifers eventually became buried in silt and clay which, although not watertight, tended to concentrate underground percolation in the more porous aquifers.

The fact that pressure formerly was present in some of these areas sufficient to sustain artesian wells, supports the theory that the aquifers pinch out in marine clays at the lower end of the valley. If this is correct the underground storage basin is enclosed or pocketed in the lower portion of the valley where the soil is fertile and long-time demand for irrigation water can be expected.

If 10 per cent of the valley fill is water-bearing material and if 30 per cent of this consists of voids that can be used for water storage, then 3 per cent of the total volume of the valley fill can be utilized as an underground storage basin. This means that within the feasible pumping lift of 200 ft. there is available under the area included in the district, storage for ten to twenty times the total surface storage. A sidelight on this estimate is the fact that underground storage in this area actually has supplied a 40,000-acre-ft. overdraft per year for the last 20 years of subnormal rainfall, and during that time has sustained full production and property values throughout the entire area.

However, during this two-decade period of overdraft the water table went down an average of 100 ft. or more to an all-time low in November 1934. This low was reached just as the district was ready to profit from its preparations (carried on for more than 15 years) for putting water underground. Since spreading operations began, some 2,000 wells distributed over the entire valley

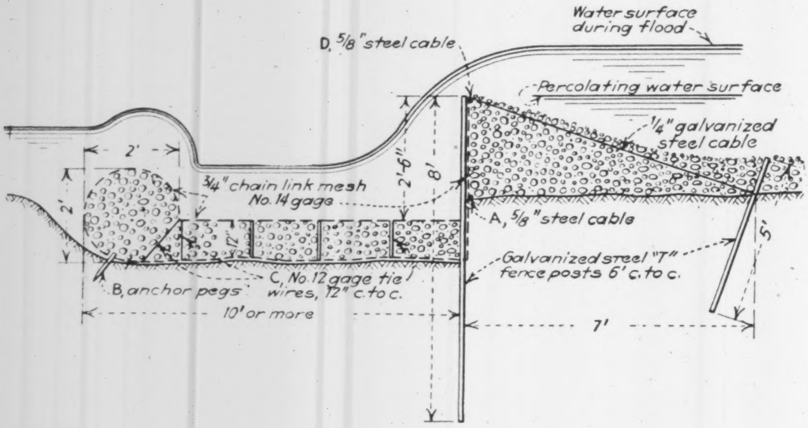

Water surface during flood
D, 5/8" steel cable
Percolating water surface
1/4" galvanized steel cable
2'
3/4" chain link mesh No. 14 gage
2'-6"
8'
2'
12"
A, 5/8" steel cable
5'
Galvanized steel "T" fence posts 6' c. to c.
C, No. 12 gage tie wires, 12" c. to c.
B, anchor pegs
10' or more
7'

Fig. 2. Design for wire sausage dams that can be built for about $7 per linear foot.

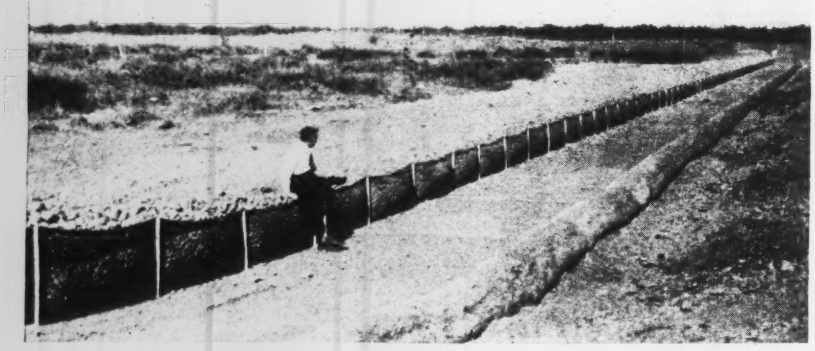


Fig. 3. Wire percolating dam on Guadalupe Creek.

have been measured regularly. From early observations it was possible to select about 25 "index" wells which are measured weekly to determine the time of the most significant high and low levels. At these times, all the wells are measured and the levels recorded.

The rise of the water table has been greatest and quickest in the upper reaches of the delta cones where streams debouch onto the valley floor. Naturally, the rises are most marked where the artificial percolating areas have been maintained on the gravel cones. However, the rises begin in the Fall before any runoffs or even before any rainfall occurs, because the cessation in the fall of seasonal pumping allows quick rebound of water levels in the wells. This represents the refilling by percolation of the localized depression around each well (ordinarily called the well cone) caused by Summer pumping.

Average fluctuation of the water table in the Santa Clara Valley is meaningless unless the points at which the fluctuation is observed are grouped and averages are taken in each group. There are three natural groupings: First, the gravel percolation areas at higher elevations on the delta cones which constitute about 12 per cent of the valley area. The water table in this zone is influenced principally by the conditions obtaining just at the time with respect to percolation from storm waters or from reservoir releases. Second, the area in which artesian wells flowed before irrigation became general, which constitutes about 26 per cent of the total. This area is influenced principally by pumping both in the area itself and in the zone above it from which the supply comes. Third, and finally, there is the central or lower portion of the valley floor, constituting about two-thirds the total area. Fig. 4 shows the location of the three areas used chiefly for percolation. Two of these, the Coyote and Guadalupe-Almaden, are supplied from large storage reservoirs which can release water to maintain stream flow on their gravel cones for many months. The graphs in Fig. 5 show that water table fluctuations in these percolating areas have a much wider range and a more rapid fluctuation, especially in the rising stages, than elsewhere.

Streamflow is measured continuously where the streams debouch from the mountains (at the heads of the gravel cones) and again below the lower limits of percolation. Beyond these lower gaging stations the flow is wasted into the Bay, hence the difference on any one stream or group of streams between measurements at upper and lower measuring stations represents the amount of deep percolation.

Methods and facilities used for getting water underground fall into three classes: natural streambed percolation; artificial streambed percolation (reservoirs) and off-channel percolation (winter irrigation).

The first of these is the most important because the natural stream channels usually have sand or gravel bottoms and in their upper reaches most of them have very extensive gravel cones, apparently with more or less direct access to the aquifers. These upper gravel areas will quickly absorb a large amount of water during the first stream flows of each season and pass it rapidly into the aquifers, which distribute it to the surrounding material in which they are imbedded. However, the limits of the gravel cones cannot be accurately determined.

Each of the principal streambeds, especially those upon which there is reservoir control, has been staked off in sections one mile long and percolation rates in each mile are measured. The rate of streambed absorption is naturally greatest at the upper end of the streams, where the material is coarser, the grade steeper and velocity is sufficient to prevent much silting. Farther down, the streams traverse marine clay deposits sufficiently impervious to prevent much seepage.

The second class of percolating works, namely, streambed percolating areas or reservoirs, have been constructed in the beds of several of the larger streams and are maintained to augment natural streambed percolation. Percolation in these

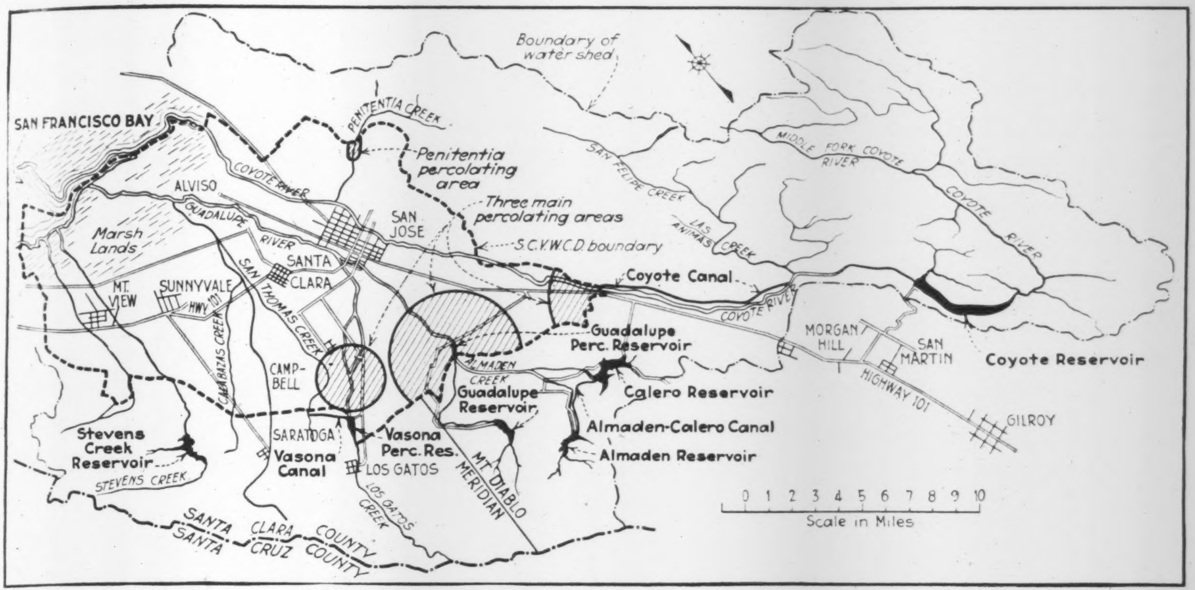

Boundary of water shed
SAN FRANCISCO BAY
PENITENTIA CREEK
Penitentia percolating area
Three main percolating areas
COYOTE RIVER
ALVISO
GUADALUPE RIVER
SAN JOSE
S.C.V.W.C.D. boundary
Marsh Lands
SANTA CLARA
MT. VIEW
SUNNYVALE
HWY. 101
SAN THOMAS CREEK
CALABAZAS CREEK
CAMP-BELL
Coyote Canal
Guadalupe Perc. Reservoir
ALMADEN CREEK
Guadalupe Reservoir
Calero Reservoir
Almaden-Calero Canal
Almaden Reservoir
SARATOGA
Vasona Canal
Vasona Perc. Res.
LOS GATOS
LOS GATOS CREEK
MT. DIABLO MERIDIAN
Stevens Creek Reservoir
STEVENS CREEK
SANTA CLARA COUNTY
SANTA CRUZ COUNTY
SAN FELIPE CREEK
LAS ANIMAS CREEK
MIDDLE FORK COYOTE RIVER
COYOTE RIVER
MORGAN HILL
SAN MARTIN
HIGHWAY 101
GILROY
Coyote Reservoir
0 1 2 3 4 5 6 7 8 9 10
Scale in Miles

Fig. 4. Map showing location of three main percolating areas.

areas, is determined by gagings. The third class or off-channel percolation, is in all respects an irrigation operation with the objective of deep percolation. Typical results on some of this winter irrigation in two seasons are shown in Table I.

Typical percolation reservoirs

To augment natural channel percolation, favorable streambed areas on some of the principal streams have been made into artificial percolation areas by creating reservoirs in the stream channels themselves. To be favorable to this purpose the topography must permit submergence of maximum area by a dam of minimum cost. A second requirement is that current meter measurements show high rates of percolation at low flows. Such locations, for the most part, are well up toward the sides of the valley where the surface material is coarser and more open.

There are two large percolating reservoirs. One is just below the lower gorge of Coyote Creek, where the river passes through a very narrow section of the valley entirely filled with rather coarse gravel, below which a well-developed gravel cone has been formed. The second is at the junction of Guadalupe and Almaden creeks. Above these two percolating areas is a total of 45,000 acre-ft. of surface storage which probably will be doubled later on. This should enable percolating reservoirs below to be operated most of the summer and fall seasons.

The Coyote Creek percolating reservoir was formed by a dam 10 ft. high and 150 ft. long, with a movable or collapsible super-structure (Fig. 1). It is not used when the stream flow is muddy; the operating plan is to remove the collapsible super-structure before winter floods come and thus to prevent the reservoir from being silted up. When winter flow has subsided to a point where it is reasonably clear the dam super-structure is raised and desilted reservoir releases are spread for percolation.

Raising the super-structure to detain water is permissible only when the silt content is low enough to show not more than one part in 2,000 of silt settling out in ten minutes. This test is made in a specially graduated glass cone holding about one quart. (Turbidity can be determined automatically by a photoelectric cell.)

The Guadalupe-Almaden reservoir is operated at several different levels with downstream portions progressively lower by 2 to 3 ft. It is formed by the construction of very simple and inexpensive wire sausage dams which should not be more than 2 or 3 ft. high (Figs. 2 and 3), built in series in the streambed. Percolation resultant from such structures is close to a direct function of the area submerged, hence they afford a considerable measure of control of percolating areas. They can be built with but little skilled labor at a cost of about $7 per ft.

In constructing the wire percolating dam shown in section in Fig. 2, one edge of the wire mesh is fastened to the steel cable marked *A*, the mesh is laid across the prepared bottom to *B* where anchor pegs are driven to hold it while it is pulled up around subsequently placed gravel and then is fastened down by the numerous tie wires marked *C*. Finally it is brought up along the steel fence posts to be fastened to the top cable *D*. The mesh is obtainable in widths just sufficient to meet these requirements. Wire diameters and mesh size are specified as desired, heavy for coarse gravel and lighter for fine gravel.

Off-channel percolation

Off-channel percolating works for winter irrigation are used chiefly in upper portions of the valley where gravel deposits are extensive. Diversion from main streams to the off-channel gravels is accomplished by low diversion dams, canals and distributing works. The streams are so flashy that where there are no large detention reservoirs much additional water can be conserved by small amounts of storage at these diversion dams, which are emptied and filled with each storm, sometimes half a dozen or more times in one season.

TABLE I—TYPICAL PERCOLATION RESULTS FOR 1936 AND 1937

Basin	Total water entering valley floor (acre-ft.)	Percolation in reservoirs (acre-ft.)	Off-channel percolation (acre ft.)	Wastage into tide water (acre-ft.)	Natural* streambed percolation (acre-ft.)	Length of streambed affected (miles)	Seasonal streambed consumption (acre-ft. per mile)
1935-36							
Coyote	54,310	5,000	0	26,230	23,080	14	[illegible]
Penitencia	4,463	0	2,928				
Los Gatos	24,580		5,530	7,200	19,038	18.5	[illegible]
Guad.-Almaden	16,180	6,000	1,500				
1936-37							
Coyote	62,850	17,445	0	33,460	11,945		
Penitencia	6,900	0	1,593	3,164	2,143	2.5	
Los Gatos	33,340	0	10,962				
Guad.-Almaden	26,540	7,476	1,216	22,010	7,264		

* Figures in this column are those in column two less the sum of those in columns three, four and five.

TABLE II—EXAMPLES OF OFF-STREAM PERCOLATION RESULTS

Location	Type of soil	Season	Amount of percolation (acre-ft.)	Equivalent depth of coverage (ft.)	Area submerged (acres)
Almaden vineyard	Gravelly vineyard ..	1937	1,216	.....	.....
Penitentia creek	Abandoned R. R. across gravel cone.	1936	2,980	314	9.5*
Penitentia creek	Abandoned R. R. across gravel cone.	1937	1,593	168	9.5

* After two weeks of continuous operation the upper portion of this area was absorbing water at the rate of 7.3 ft. per day.

TABLE III—EXAMPLES OF PERCOLATING RESULTS IN TWO STREAMBED RESERVOIRS

Location	Area covered (acres)	Season	Period of use (days)	Amount of percolation (acre-ft.)	Percolation rate (ft. per day*)
Coyote Creek Gorge	32	1936	87	5,000	1.8
Coyote Creek Gorge	32	1937	199	14,005	2.2
Guadalupe-Almaden	17	1936		6,000	2.2
Guadalupe-Almaden	17	1937		7,476	2.0

* The rate is expressed in terms of acre feet per acre per day or "feet per day."

TABLE IV—TOTAL ARTIFICIAL PERCOLATION FOR 1937

(From report of resident engineer J. W. Hunt)

Percolation Works	Absorption (acre-ft.)	Per cent of total
Old Page Ditch	8,616	20.2
New Page Ditch	655	1.5
Masson Ditch	1,216	2.8
Penitencia Creek Ponds	1,458	3.4
Total off-channel percolation	11,945	28.0
Guadalupe-Almaden Percolation Reservoir	7,476	17.5
Coyote Percolation Reservoir	16,725	39.2
Total Artificial Streambed	24,201	56.7
Stevens Creek Ponds	1,876	4.4
Coyote River Ponds	4,563	10.7
Total Temporary Streambed Checks	6,439	15.0
Foster Well	135	0.3
Total Artificial Percolation	42,720	100.00

TABLE V—PROGRESSIVE RISE IN AVERAGE (?) WATER TABLE LEVELS

(Elevations in feet above sea level)

	Period of the year	1934	1935	1936	1937
Percolation areas	High		145	155	156
	Low	95	113	115	135
Former artesian area	High		6	5	7
	Low	9	19	21	21
Remainder of the district	High		34	43	50
	Low	19	26	28	41
Average over entire district	High		62	68	71
	Low	35	40	41	52

Fig. 5. Water table fluctuations in the three areas of the conservation district

Old diversion ditches have been acquired by the district here and there and rebuilt to convey flood water to desilting basins for clarification before it is used for the winter irrigation. A very considerable amount of water is put underground by such devices; in places irrigated continuously during the season of natural stream flow (100 days or more) the amount may be more than 200 acre-ft. per acre. Measured rates of absorption in small areas of this sort have been as high as 4 to 5½ ft. per day. For larger areas the rates are shown in Table III.

Diversions for winter irrigation from two old reconstructed ditches and the new Vasona Canal totaled 5,530 acre-ft. during 1936 and 10,962 acre-ft. in 1937. Most of this water undoubtedly passed directly into underground storage, as it was spread on highly absorbent areas of great depth and at a time when there was little evaporation or transpiration. Such portions as did not quickly sink below the range of the tree roots, but instead were stored in surface layers of soil, would be available for consumption by the trees next season where it lessens correspondingly their normal draft.

Results of replenishment program

By varying the rate of reservoir releases the point where the water goes underground can be controlled to a considerable degree. The larger the quantity of flow from the reservoirs the farther downstream it will go and the deeper the underground penetration. Hence, if water levels lag in wells in the lower part of the valley, the indicated remedy is to increase the rate of reservoir releases, or, if there are favorable gravel beds of porous areas near the lower end of the valley, percolating reservoirs or off-channel percolating works may be constructed there.

The total amount of water put to

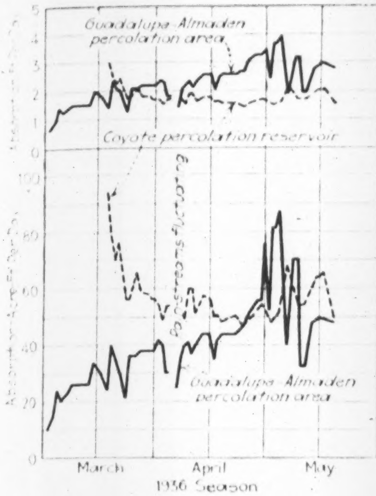


Fig. 6. **Rates of percolation in two large spreading areas.**

beneficial use in 1937 (a normal rainfall and runoff season) was about 102,000 acre-ft. or about 60 per cent of the total runoff. About 42 per cent of this (Table IV) was passed underground through the use of artificial works. Fig. 5 shows the average rise in the several zones for several years.

From an all-time low in 1934 to January 1, 1938, the water table has risen more than 100 ft. in places near the stream percolation works. In the main or central portion of the valley the average annual rise has been rather uniform. The total has been more than 30 ft., of which nearly one-half occurred during 1937. This was practically a normal year and the first year in which all of the district's works were in use.

During 1937 reservoir releases on the principal streams made it possible to supply the percolation areas continuously during the summer, and as a result percolation through the radiating aquifers which rated as a distributing system, held up the water levels of the wells in the central portion of the valley, practically eliminating the usual summer drop resultant from irrigation pumping. This appears to make the spring rise a permanent gain. The additional pressure caused by the higher water levels in the central part of the valley has also caused the old artesian basin to rise sharply (more than 10 ft.) during the year. Altogether the valley's underground water storage has actually been replenished in average years about three times as fast as it had been going down.

Fresno's Garbage Plan Succeeds

THE SANITARY FILL METHOD of garbage disposal and the low-body motor trucks specially designed to reduce the cost of garbage collection at Fresno, Calif., have continued to be as successful as was reported in the article published in *Engineering News-Record* April 25, 1935, p. 592.

A Fresno city official stated, when interviewed by a representative of this journal recently: "In the three years since this equipment was put in service we have made no replacements, and although money has been laid aside in the budget for this fiscal year to replace five of the 1½-ton motor trucks (provided with special bodies for hauling garbage) the equipment shows no indication of requiring replacement. We keep a very close cost account on all these trucks and other equipment, including not only cost of operation but also maintenance, depreciation and overhead. With depreciation at $240 per year and overhead at $165.51 per year, the total average cost per mile for the past year was 6 c. per truck and some have run as low as 5.4 c. Under these conditions the department has operated so economically that it has been possible to reduce the garbage collection rates paid by householders three times in the three years the city has maintained this service for its residents.

"The total quantity of garbage hauled in the past year was 76,894 cu. yd. and using the unit of weight we have found to be near a correct average the weight equivalent would be 17,301 tons. For this quantity the entire disposal cost was $5,103.81 or 29 c. per ton. This includes the cut-and-cover method of disposal described and illustrated in the article mentioned in the first paragraph of this article.

"A noteworthy feature of the service rendered by the city's waste disposal department is the hauling away and disposal of rubbish without additional cost to the householder. Household rubbish, old newspapers, garden trimmings and other waste accumulated at residences totaling 51,627 cu. yd. has been hauled away and disposed of by this department during the past year on a regular pickup schedule made once or twice a month. Cost of this operation is included in the garbage disposal figures given."

The Fresno waste disposal department is under the direction of Jean L. Vincenz, commissioner of Public Works.

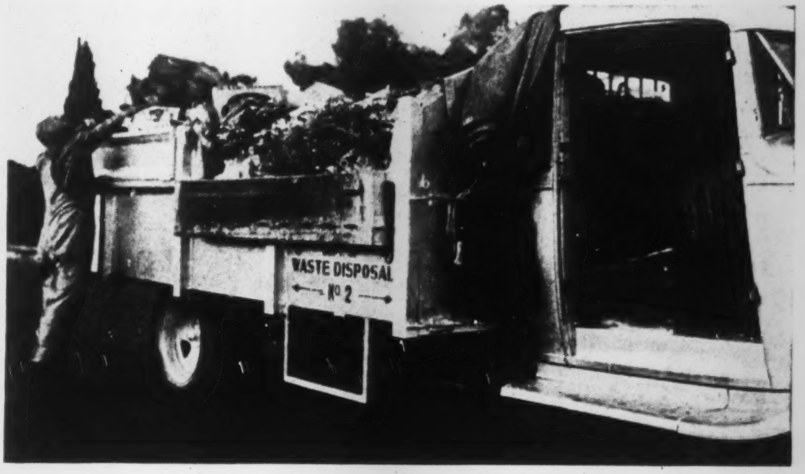


Fig. 1. **Fresno garbage truck of special body design which enables scavengers to load from street level as shown. Metal frame below truck body in foreground is for carrying the scavenger's tub.**

Interpreting Data from Strain Rosettes

by George E. Beggs and Elmer K. Timby

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A practical and rapid procedure for deducing stresses from strain-gage measurements on four gage lines on a plate subjected to stress

Determination of the principal stresses and strains on a plate subjected to strain in two dimensions by taking tensometer measurements on rosettes of intersecting gage lines has been practiced only in the most scientific of investigations. More frequent use of the procedure is desirable, and this article by Professors Beggs and Timby setting forth the mathematical theory of strain rosette measurements and a form for recording the data is a distinct aid to that end. Important pioneer strain rosette work in this country was done two decades ago by Dr. William Hovgaard in connection with ship design. The Stevenson Creek Dam investigation of ten years ago and the current steel rigid frame studies at the Bureau of Standards represent other examples of rosette work, to which should be added various academic studies in university laboratories. In their work, Professors Beggs and Timby developed a technique for making the strain measurements which is as important to the success of the procedure as interpretation of the results. An article presenting this technique will be published next week.—Editor.

In a recent test of a structural model it was required to determine the stresses and strains at each of a number of points. This was accomplished by means of strain measurements made on four 1-in. gage lines spaced 45 deg. apart and intersecting at the points. As diameters of 1-in. circles, the four gage lines were designated as rosettes, and always had the orientation shown in Fig. 3. Huggenberger tensometers were used to measure the strains, whose interpretation in terms of design data is the subject of this article. The advantages of measuring four gage lines instead of three have already been pointed out by Dr. William Hovgaard and Dr. William R. Osgood, in the references cited at the bottom of the next page, and to them the writers are indebted for some of the ideas herein.

The plating at the location of the rosette was considered as a thin sheet of elastic material of indefinite extent subjected to unknown coplanar stresses when the model was loaded. With four measured strains taken on the four axes of a rosette as basic data the problem was to write formulas for:

I. The four direct stresses (s_0, s_{90}, s_{45}, and s_{135}) parallel to the four axes respectively.

II. The shear stresses (q_0, q_{90}, q_{45}, and q_{135}) parallel to the four axes respectively.

III. The principal direct stresses (s_5, for maximum algebraic value and s_6, for minimum algebraic value).

IV. The principal shear stresses ($+q$ maximum and $-q$ minimum).

V. The maximum internal shear stress in the plate.

VI. The principal direct strains (e_5 for maximum and e_6 for minimum algebraic value).

VII. The principal shear strains ($+y$ maximum and $-y$ minimum).

VIII. The directions of the principal direct stresses and strains. (One value of angle, Θ_{max}, for the maximum algebraic value of stress and strain and another angle, Θ_{min}, for the minimum algebraic value, of stress and strain, both measured counterclockwise from the 0 deg. axis and both less than 180 deg.).

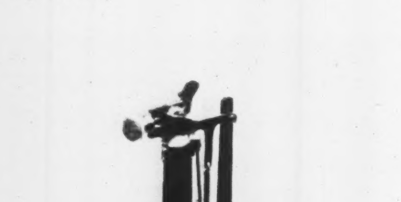


Fig. 1. **Typical setup of tensometer over rosette of four intersecting diameters of a 1-in. circle.**

It was considered desirable to use a computation form that would provide a systematic calculation of the quantities listed under items I to VIII above. A copy of this form, together with a partial explanation of its use, is a part of this article. (Fig. 2.)

In deriving the necessary formulas the following notation, in addition to that cited above and on the computation sheet, was used:

e_n = unit strain in any direction n.

ε_n = 1000 (e_n); notation adopted to reduce the number of zeros in decimal values.

s_n = unit direct stress acting in direction n, that is perpendicular to the surface of a plane perpendicular to direction n, in lb. per sq. in.

Θ_s = direction of principal shear stress, q, measured counterclockwise from the 0-deg. axis and less than 180 deg.

q_n = unit shearing stress acting in direction n, that is on a plane parallel to direction n, in lb. per sq. in.

LINE	FUNCTION	FORMULA	ROSETTES 1	2	3	4	5
1	ϵ_0		1000 times the unit strain observed on the respective axes				
2	ϵ_{90}						
3	ϵ_{45}						
4	ϵ_{135}						
5		0.3 (1)	Constant, 0.3, is Poisson's Ratio (m)				
6		0.3 (2)					
7		0.3 (3)					
8		0.3 (4)					
9		(1 + 6)					
10		(2 + 5)					
11		(3 + 8)					
12		(4 + 7)					
13	S_0	31870 (9)	Constant, 31870, is $\frac{E}{1000\ (1-m^2)}$ S_0, etc., are the unit direct stresses parallel to the respective axes.				
14	S_{90}	31870 (10)					
15	S_{45}	31870 (11)					
16	S_{135}	31870 (12)					
17		(15 - 16)					
18		(14 - 13)					
19	q_0	0.5 (17)	← Unit shear on 0° axis. $q_{90} = -q_0$				
20	q_{45}	0.5 (18)	← Unit shear on 45° axis. $q_{135} = -q_{45}$				
21		$(17)^2$					
22		$(18)^2$					
23		(21 + 22)					
24		$\sqrt{23}$					
25	q	± 0.5 (24)	← "Principal" unit shear stress, located on planes making 45° with the principal stresses.				
26		(13 + 14 + 15 + 16)					
27		0.25 (26)					
28	S_5	(27 + 25)	← One principal unit direct stress — (max).***				
29	S_6	(27 - 25)	← Other principal unit direct stress — (min).***				
30	MAX. INTERNAL SHEAR		← Maximum unit shear stress in the plate at the point. Located either on a plane normal to the plate at an angle of 45° to the principal stresses, or on a plane at 45° to the surface of the plate.				
31		0.3 (28)**					
32		0.3 (29)**					
33		(28 - 32)					
34		(29 - 31)					
35	ϵ_5	0.0000345 (33)	← *1000 times one Prin. Unit Strain — (max).***				
36	ϵ_6	0.0000345 (34)	← *1000 times other Prin. Unit Strain — (min).***				
37	γ	± (35 - 36)	← 1000 times Prin. Unit Shear Strain — parallel to a plane making 45° with the Principal Strains.				
38	Tan. 2θ	$-(17 \div 18)$					
39	2θ	From table					
40	2θ	(39) + 180°					
41	θ FOR MAX. STRAIN		← The counterclockwise angle from 0° axis to S_5 (less than 180°)				
42	θ FOR MIN. STRAIN						

Notes: All unit stresses are in lb. per square inch.
Numerals within parentheses or radicals are line numbers.
Direct stress is considered positive when tensile.
Shear stresses are positive when acting in a clockwise direction if viewed directly
* The constant, 0.0000345 is 1000 divided by E.
** The constant, 0.3 is Poisson's Ratio.
*** The maximum and minimum algebraic values.
E = 29,000,000 lb/sq. in.

Fig. 2. Outline of computation form used to convert observed strains into design data. Explanations beside lines of the form are developed in the text.

Published papers referred to in this Article: "Determination of Stresses in Plating from Strain Measurements," Paper No. 1, 39th General Meeting, Society of Naval Architects and Marine Engineers, N. Y. City, November 19, 1931, by Dr. William Hovgaard.

"Determination of Principal Stresses from Strains on Four Intersecting Gage Lines 45° Apart," Journal of Research, Bureau of Standards, No. 6, December 1935, pp. 579-581, by Dr. William R. Osgood.

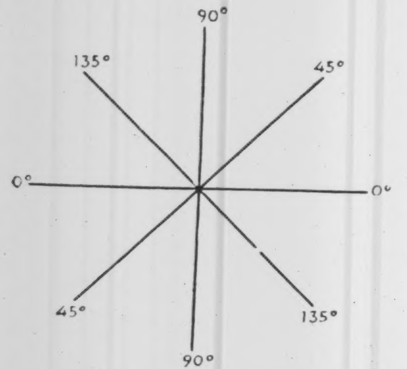


Fig. 3. Arrangement of four gage lines that make up a rosette.

γ = principal unit shearing strain.

E = linear modulus of elasticity in lb. per sq. in.

E_s = modulus of elasticity in shear, in lb. per sq. in.

m = Poisson's ratio, ratio of the unit strain perpendicular to a direct stress, to the unit strain parallel to the direct stress, both strains being entirely a result of a single direct stress.

Convention of signs

Direct strains and stresses are taken as positive (+) when tensile, while shearing strains and stresses are positive (+) when the shearing couple has a clockwise direction. In the following presentation the formulas or equations incident to the derivation are followed by a letter in parentheses. Formulas for quantities listed under items I to VIII are indicated by the line number on which they occur on the computation form.

By moments of forces about a corner of the plate element shown in Fig. 4, (or of a rectangular element): $q_0 = -q_{90}$ and $q_{45} = -q_{135}$. Consequently only q_0 and q_{45} are listed on the computation form. Also in the following derivation of formulas, q_0 will be used for either q_0 or $-q_{90}$ and similarly q_{45} will be used for either q_{45} or $-q_{135}$.

Stresses along gage lines

From elementary applied mechanics, equations for unit strain may be written as follows:

$$\frac{\epsilon_0}{1000} = \frac{s_0}{E} - m\frac{s_{90}}{E} \qquad \text{(a)}$$

$$\frac{\epsilon_{90}}{1000} = \frac{s_{90}}{E} - m\frac{s_0}{E} \qquad \text{(b)}$$

Solving Equations (a) and (b) simultaneously gives,

$$s_0 = \frac{E}{1000(1-m^2)}(\epsilon_0 + m\epsilon_{90}) \qquad (13)$$

$$s_{90} = \frac{E}{1000(1-m^2)}(\epsilon_{90} + m\epsilon_0) \qquad (14)$$

Similarly

$$s_{45} = \frac{E}{1000(1-m^2)}(\epsilon_{45} + m\epsilon_{135}) \qquad (15)$$

$$s_{135} = \frac{E}{1000(1-m^2)}(\epsilon_{135} + m\epsilon_{45}) \qquad (16)$$

In the derivation of the formula for q_0 (Eq. 19 below) it is assumed that the element shown in Fig. 4 for the 45-deg. and 135-deg. stresses is split along the diagonal MN, and that the element is a 1-in. cube, subjected to two dimensional stress as shown. That portion of the element which is above the line MN is considered as a free body. A resolution of forces is made parallel to the diagonal MN; that is, parallel to q_0. The numerically equal streses q_{45} and q_{135} have components, parallel to MN, which are equal in magnitude and opposite in direction. Therefore q_{45} and q_{135} need not appear in the equilibrium equation

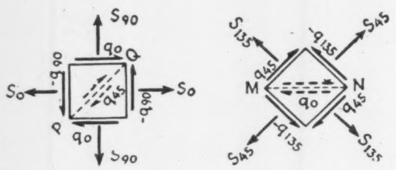


Fig. 4. How the direct and shearing unit stresses act on the elements of a plate.

$$q_0\sqrt{2} = s_{45}\frac{1}{\sqrt{2}} - s_{135}\frac{1}{\sqrt{2}} \qquad \text{(c)}$$

or

$$q_0 = \frac{1}{2}(s_{45} - s_{135}) \qquad (19)$$

If the element shown in Fig. 4 for the 0-deg and 90-deg. forces is also assumed to be a 1-in. cube, split along the diagonal PQ, then, by resolution of forces parallel to PQ,

$$q_{45} = \frac{1}{2}(s_{90} - s_0) \qquad (20)$$

Next considering the unit stresses acting on a triangular portion of the rectangular element in Fig. 5, a resolution of forces parallel to the direction of q_n may be expressed by the equilibrium equation which follows:

$$q_n\,ds = s_0\,dy\cos\Theta - s_{90}\,dx\sin\Theta - q_0\,dx\cos\Theta + q_0\,dy\sin\Theta \ldots \text{(d)}$$

in which $-q_{90}$ is replaced by q_0.

Dividing (d) by ds, and substituting

$$\frac{dy}{ds} = \sin\Theta, \text{ and } \frac{dx}{ds} = \cos\Theta$$

$$q_n = s_0\sin\Theta\cos\Theta - s_{90}\sin\Theta\cos\Theta + q_0(\sin^2\Theta - \cos^2\Theta) \ldots \text{(e)}$$

To find the maximum and minimum value of q_n, and the angle Θ_s at which it occurs, q_n is differentiated with respect to Θ and the derivative equated to zero. The function of Θ resulting defines $\Theta_{s\,max}$ and $\Theta_{s\,min}$, and these values substituted in (e) furnish the equations for the maximum and minimum principal shear stresses, $+q$ and $-q$ respectively. These operations are readily performed if (e) is first written in terms of the double angle

$$q_n = \frac{1}{2}(s_0 - s_{90})\sin 2\Theta - q_0\cos 2\Theta \qquad \text{(f)}$$

Differentiating

$$\frac{dq_n}{d\Theta_s} = (s_0 - s_{90})\cos 2\Theta + 2q_0\sin 2\Theta \qquad \text{(g)}$$

Equating the derivative to zero the following function of Θ is obtained

$$\tan 2\Theta_s = \frac{-(s_0 - s_{90})}{2q_0} \qquad \text{(h)}$$

From (h) it is possible to write

$$\sin 2\Theta_s = \pm\frac{(s_0 - s_{90})}{\sqrt{(s_0 - s_{90})^2 + (2q_0)^2}} \qquad \text{(i)}$$

$$\cos 2\Theta_s = \mp\frac{2q_0}{\sqrt{(s_0 - s_{90})^2 + (2q_0)^2}}$$

which, when substituted in (f), give the equations for the maximum and minimum shears, hereafter designated as q, the "principal" unit shear stress

$$q = \pm\frac{1}{2}\sqrt{(s_0 - s_{90})^2 + (2q_0)^2} \qquad \text{(j)}$$

From (19) it is evident that (j) may be written as

$$q = \pm\frac{1}{2}\sqrt{(s_0 - s_{90})^2 + (s_{45} - s_{135})^2} \qquad (25)$$

Principal direct stresses

The principal direct stresses are determined in a manner similar to that just described for the determination of the principal shear stress. If the forces on the triangular element are resolved parallel to s_n (Fig. 5) and if operations are performed on the

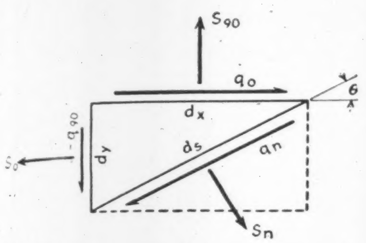


Fig. 5. **Stresses acting on a rectangular element of a plate.**

resulting equation of equilibrium similar to those which were performed on (d) it will be found that

$$\tan 2\theta = \frac{2q_0}{(s_0 - s_{90})} \qquad (k)$$

It will be noted that the two values of θ_s given by (h) lie midway between the two values of θ given by (k). Therefore the principal unit shear planes are at 45 deg. with the principal direct stresses.

The equation for the maximum and minimum direct stresses will be

$$s_n = \frac{1}{2}(s_0 + s_{90}) \pm \frac{1}{2}\sqrt{(s_0 - s_{90})^2 + (2q_0)^2} \qquad (m)$$

From the relation expressed in (19) it is possible to rewrite (k) and (m) as (38) and (o) respectively.

$$\tan 2\theta = \frac{(s_{45} - s_{135})}{(s_0 - s_{90})} \qquad (38)$$

$$s_n = \frac{1}{2}(s_0 + s_{90}) \pm \frac{1}{2}\sqrt{(s_0 - s_{90})^2 + (s_{45} - s_{135})^2} \qquad (o)$$

The sum of the direct stresses in the 0-deg. and 90-deg. directions is equal to the sum of the direct stresses in the 45-deg. and 135-deg. directions. This may be shown as follows:

Consider a cubical element of material with unit dimensions and oriented with sides parallel to the 0-deg. and 90-deg. directions, split along the diagonal parallel to the 45-deg. direction. (Fig. 4.) A resolution of forces normal to the diagonal gives

$$s_{135} = \frac{1}{2}(s_0 + s_{90}) - q_0 \qquad (p)$$

If the cube is split along the diagonal parallel to the 135-deg. direction, and a resolution of forces made normal to this diagonal, it will be found that

$$s_{45} = \frac{1}{2}(s_0 + s_{90}) + q_0 \qquad (r)$$

Adding (p) and (r) provides the desired relation

$$s_{45} + s_{135} = s_0 + s_{90} \qquad (t)$$

It was believed that a better value of s_n would be obtained if this relation was used as a means of incorporating a greater number of the experimental observations in the equations for the calculation of the principal stress. With this substitution equation (o) becomes

$$s_n = \frac{1}{4}(s_0 + s_{90} + s_{45} + s_{135}) \pm \frac{1}{2}\sqrt{(s_0 - s_{90})^2 + (s_{45} - s_{135})^2} \qquad (u)$$

Since s_5 was designated as the maximum principal direct stress, and s_6 as the minimum principal direct stress. and further, since the second term on the right of (u) equals *q*, from (25).

$$s_5 = \frac{1}{4}(s_0 + s_{90} + s_{45} + s_{135}) + q \qquad (28)$$

$$s_6 = \frac{1}{4}(s_0 + s_{90} + s_{45} + s_{135}) - q \qquad (29)$$

Principal strains

Equations for the principal direct strains may be written at once in terms of the principal direct stresses, the modulus of linear elasticity, and Poisson's ratio

$$\epsilon_5 = \frac{e_5}{1000} = \frac{s_5}{E} - m\frac{s_6}{E} \qquad (v)$$

$$\epsilon_6 = \frac{e_6}{1000} = \frac{s_6}{E} - m\frac{s_5}{E} \qquad (w)$$

and solving (v) and (w) for 1000 times the unit strain in each case gives

$$e_5 = \frac{1000}{E}(s_5 - ms_6) \qquad (35)$$

$$e_6 = \frac{1000}{E}(s_6 - ms_5) \qquad (36)$$

By definition, the principal unit shear strain is

$$y = \frac{q}{E_s} \qquad (x)$$

The relation between the linear modulus of elasticity and the modulus of elasticity in shear is contained in the following equation (see any book on applied mechanics of materials) the derivation of which is dependent upon a consideration of the linear and shearing deformations of an element of a plate subjected to pure shear

$$E_s = \frac{E}{2(1+m)} \qquad (z)$$

A relation between *q* and the principal direct stresses. s_5 and s_6. is obtained by subtracting (29) from (28)

$$q = \frac{1}{2}(s_5 - s_6) \qquad (aa)$$

Substitution of (z) and (aa) in (x) gives the equation

$$y = \frac{s_5 - s_6 + ms_5 - ms_6}{E} \qquad (ab)$$

in which y is the principal unit shear strain. The expression in which Y is 1,000 times the principal unit shear strain is

$$Y = \frac{1000}{E}(s_5 - s_6 + ms_5 - ms_6) \qquad (37)$$

The "max. internal shear" in line 30 on the computation form is defined as the maximum shear that existed anywhere in the plate, either on a plane normal to the plate at an angle of 45 deg. to the principal stresses, or on a plane at 45 deg. to the surface of the plate. Accordingly, the value entered in line 30 is the same as line (25) if the values in lines (28) and (29) are of opposite signs; but if these values have the same sign the value in line (30) is entered as one-half of the numerically larger of the two values in lines (28) and (29).

The value entered in line (41), "θ for maximum strain," is either one-half of the value in line (39) or one-half of the value in line (40). The choice between these two values is dependent upon the following fact:

Two adjacent measured strains of a rosette are each algebraically greater than either of the other two measured strains, and the axis of the algebraically maximum principal strain is necessarily between the pair of algebraically greater measured strains. This is the axis for which "θ for maximum strain" is recorded in line (41). The value in the next line, (42) is a positive angle, 90 deg. different from that in line (41), either larger or smaller, but always less than 180 deg., and corresponds to the θ for minimum strain.

The convenience and accuracy of the computation form described in this article have been verified by use in the computation of several hundred rosettes. The complete computation of the values on the forty-two lines of the computation form requires not more than 6 minutes time of an experi-

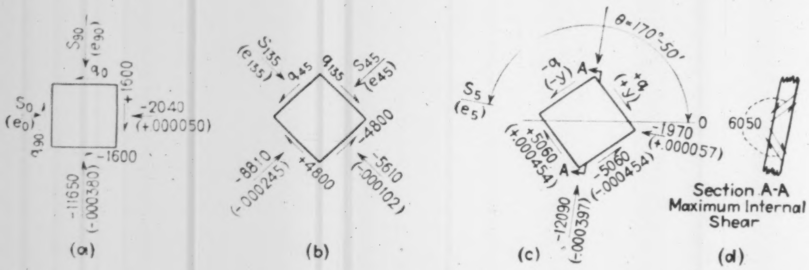


Fig. 6. Typical computed stresses and strains pictured in relation to a given rosette on which the strain measurements were made.

enced computer using a slide rule or calculating machine. Rosette computations made by this and by other published methods have been found to give the same numerical results.

The strains and stresses computed on the above form are pictured in relation to the strain rosette on the plating, in Fig. 6, coordinate strains and stresses being shown in (a) and (b) and the principal strains and stresses in (c). When such results for many rosettes are plotted on a drawing of the measured structure, it becomes possible to draw lines showing flow of stresses and strains throughout the material and thus to reach valuable conclusions on structural action.

Next week, an article by the same authors will present the technique of making strain rosette measurements.

Sewer Stoppages Cut 90 per cent

GREAT IMPROVEMENT in maintaining Los Angeles sewers, particularly with reference to stoppages, has resulted from the adoption of a plan for continuous, routine inspection instead of attention only when trouble was reported. The present program is to make an inspection at all sewer manholes regularly every 6 mo. and where conditions are bad to put in intermediate inspections at 3-mo. intervals. Whenever abnormal conditions are noted, investigation is made at once to determine and remedy the cause before there are serious results. The results attending this plan become apparent from a comparison: Ten years ago, at a time when the total mileage of sewer mains was 1,989, a typical figure was 1,253 sewer stoppages in a 12-mo. period. Now, with about 3,000 mi. of sewers but with continuous routine inspection and maintenance, the stoppages per year have decreased below 100.

Under the old method of removing obstructions a hand-operated, jointed sewer rod was used to prod and shove the foreign material in the line. A severe limitation under this method was the slow progress resultant from the use of man power under adverse working conditions. Stoppages resultant from grease, industrial wastes, roots, and rocks or sand required anywhere from two hours to several days to clear and sometimes necessitated excavation to expose the pipe.

Under the hand-labor method a crew of four men, even when working in sewers that were rodded frequently, and when cleaning only small amounts of roots, usually could not rod more than 800 to 1,000 ft. per day. This was little more than half the rate of progress now possible under a new plan of operation which uses a power-operated machine and a crew of three men.

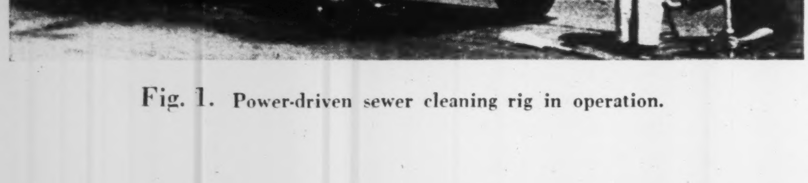


Fig. 1. Power-driven sewer cleaning rig in operation.

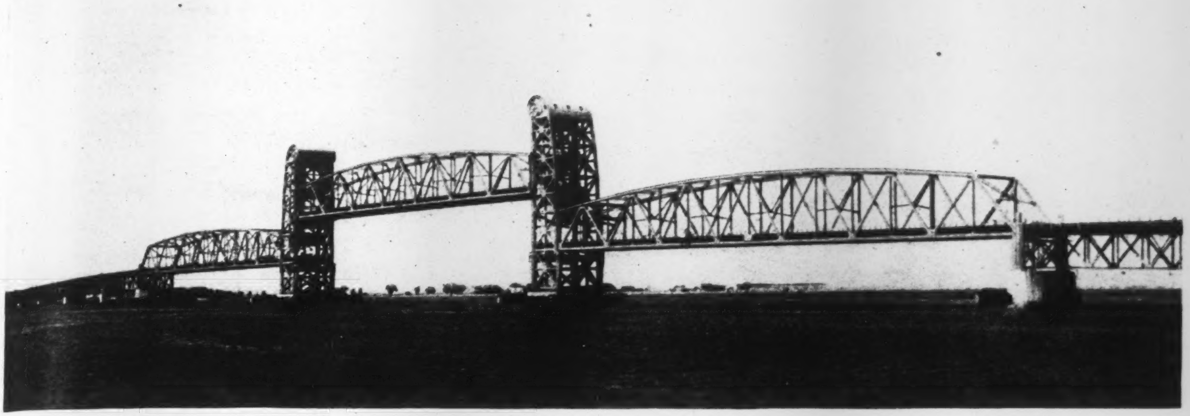
Under the new program a 2-in. spring-steel rod is thrust through from one manhole to the next and then is reeled back and forth, thrusting and pulling tools of various sorts until the obstruction is cut out and cleared away. The spring steel rod enters the sewer line through a pipe with smooth bends which is inserted in the manhole to guide the rod into the sewer and start it longitudinally into the section to be cleaned. The rod revolves slowly at the same time that it is thrust forward or pulled backward.

Where roots, grease or sand deposits are heavy, slow rates of cutting are used; where bad joints occur care has to be taken to prevent breaking the pipe. In some cases sewers are nearly closed by hard circumferential deposits of industrial wastes that incrust the inner surface. To meet such conditions a special cutterhead has been designed and is made in two sizes. The first starts at 0 and opens the line to a 4-in. diameter. The second tool cuts from 3 in. to 7 in. and is used in 8-in. sewers.

Where sand cleaning is required, rubber sand cups following an agitator are attached to the rod; augers are used for grease, which is flushed out after being loosened.

A 500-ft. length of the spring steel rod ordinarily lasts through the cleaning of about 25,000 lin.-ft. of sewer, a cost for the rod of less than half a cent per thousand feet of sewer cleaned.

Sewer maintenance is under the supervision of H. P. Cortelyou, deputy city engineer in charge of waste disposal. Lloyd Aldrich is city engineer of Los Angeles.



LONGEST HIGHWAY LIFT-SPAN RISES 95 FT. FOR JAMAICA BAY SHIPPING AT NEW YORK. IT IS 540 FT. LONG

Erecting the Marine Parkway Bridge

by W. K. Greene and E. E. McKeen
Assistant Engineers, American Bridge Co., New York, N. Y.

Fourteen of 15 spans in New York City structure assembled on barges and floated into place. Bridge includes longest highway lift-span

Spanning Rockaway Inlet in New York City is a new highway crossing that literally floated into place, for of the fifteen spans in the Marine Parkway Bridge, fourteen were erected on car-floats or barges and, on them, moved to their final positions in the structure. Free use was also made of floating equipment in erecting the towers for a 540-ft. lift-span, central unit in the bridge and the longest highway lift-span yet built. The erection of the superstructure of this bridge in the record time of 7½ months was made possible by the floating method of erection, and was one of the most extensive operations of its kind ever undertaken.

Marine Parkway, which passes over the new bridge, connects Flatbush Ave., Brooklyn, with highways on Rockaway peninsula and provides ready access to Jacob Riis Park on the oceanfront. The bridge, 3,839 ft. long, contains, as stated, a 540-ft. through-truss lift-span, and this is flanked on each end by a 540-ft. through truss fixed span and six deck-truss spans varying in length from 97 to 214 ft. The deck spans are arranged in a two-span continuous unit, a three-span continuous unit and a simple span. About 13,000 tons of structural steel was required, of which 3,400 tons is silicon steel, which is used in the through spans and the towers.

The crossing provides for both vehicular and pedestrian traffic on a four-lane, 44-ft. roadway and a 6-ft. sidewalk. The walk on the entire structure and the roadway on the deck spans are of reinforced concrete, while the roadway on the through spans and towers is of open-type steel grating construction.

Plan of attack

On account of the short time allowed for field operations, it was necessary to start the complicated tower and lift-span work first. In general, the plan was to erect with a derrick boat the first tier of the towers, which reached to and included the roadway. A stiffleg derrick was then erected on the north tower roadway. Two steel car-floats, 325x39 ft., braced together and fitted with steel falsework, to support the lift-span slightly higher than its final position (55 ft. above high water), were moored on the north side of this north tower. The lift-span was erected on these car floats by the stiffleg derrick on the tower, steel being brought in by barge. After being assembled, the span was floated to final position between the towers and transferred to the bridge seats.

Temporary triangular towers surmounted by stiffleg derricks were next erected, one at each end on the top chord of the lift-span, and these derricks completed erection of the towers. Finally, a mast and boom were placed on the north face of the north tower to erect first the south and then the north flanking through-truss spans in a manner similar to the procedure used for the lift-span.

Simultaneously with much of the above work, the deck spans, one by one, were erected on barges at a place remote from the bridge site and floated to final position in the structure. The only variation from the floating procedure was for the north 97-ft. span which was only partly over water and therefore had to be erected on falsework.

The car-floats used for erection of the through spans were placed 55 ft. on centers and parallel to the span

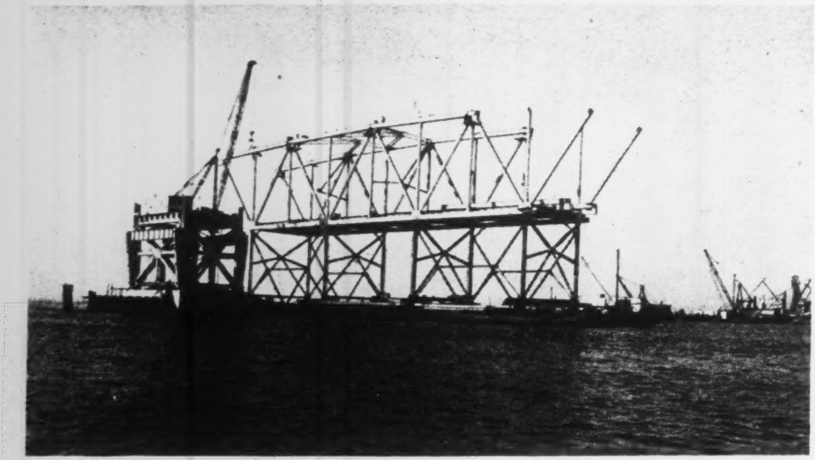


Fig. 1. The 540-ft. lift span was assembled on car-float falsework by a derrick mounted on the partly-completed tower.

to receive the 50-ft. high steel falsework which rested on three-tier beam grillages symmetrically placed. Bracing of timber and wire rope lashing secured the floats against relative lateral or longitudinal movement, but no effort was made to restrain other movement which might result from wave action. The bottom ends of the falsework columns were pivoted to provide for reasonable variations in elevations of the floats resulting from inequality of load or wave action, and at the top of the columns the span was carried in bearings so as to facilitate release of the car-float after the span was placed. The only positive fastenings between the span and the falsework were small knee braces (to take longitudinal forces) and the wire rope lashing; and these connections were released as soon as the span was in position. The same falsework and car-floats were used for all three through spans.

Since the derrick on the north tower and later the mast and boom were in fixed positions, the car-floats were moved from time to time as erection progressed, to make the work accessible. Erection proceeded both ways from the center of a span, with the maximum unbalanced condition being about two panels. The bottom chord splices and the bottom ends of the diagonals and verticals were riveted prior to floating to avoid loss of camber in the permanent structure.

The spans were erected complete on the falsework except for the floor grating, walk and handrail. One through-span ready for floating weighed 1,800 tons, and with car-floats and falsework formed a structure 540 ft. long, 94 ft. wide, and 125 ft. high, weighing 5,000 tons, which is comparable in size and tonnage to many ocean-going ships.

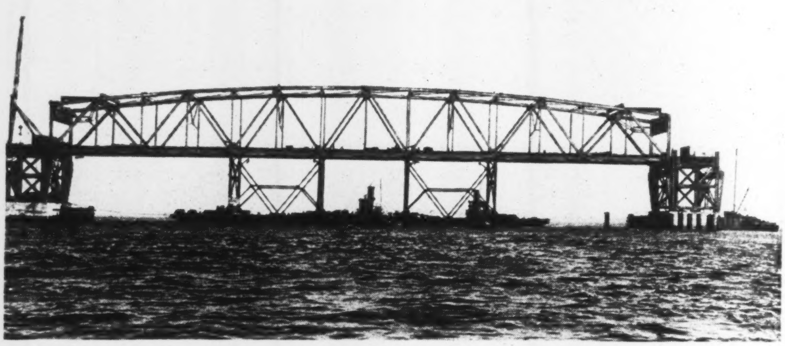


Fig. 2. The second step was to float the span into position at high tide and transfer it to the towers during an ebbing tide.

Floating-in

Rockaway Inlet at the bridge site is only a few miles from the Atlantic Ocean and has an average tide of about 5 ft. All spans were floated in at high tide using tug boats aided by anchor lines connected to air-operated winches. After the lift span, for example, was lashed in position, it was raised about 2 ft. (using hydraulic jacks set on the piers) and blocked on shims; special stiffeners were provided on the end floorbeams to permit this jacking. Then, as the tide ebbed, the load on the falsework was reduced, and finally the barges floated free, to be returned for the erection of the next span.

The total movement of the through-truss bearing points relative to water level (combined jacking of the span and ebbing of the tide) necessary to overcome the truss deflection and buoyancy of the car-floats and release the span was 2.9 ft.

Procedure for the flanking through-truss span was similar, with one exception. This relates to jacking at the ends remote from the towers where the ends of the trusses are pinned to the tops of rocker bents about 23 ft. high. At these points the jacking was done at the bottoms of the bents where a temporary guide giving both lateral and longitudinal support was installed. These guides (one for each leg of the bent) permitted rotation of the bent about the bottom end, which was necessary since the bottom chord of the truss elongated about 3 in. in changing from compression on the falsework to tension as a simple span.

Deck span procedure

Barges 114 to 186 ft. long by 34 ft. wide were used to erect the deck spans, two barges of the same length being placed side by side and lashed with wire rope bracing, as for the through spans. The loads of the deck spans were relatively small, and the falsework, which varied from 4 to 20 ft. in height, was made of timber. A grillage of steel beams carried the load from the trusses, spaced $31\frac{1}{2}$ ft. on centers, to the falsework located centrally on the barges.

Mill Basin, adjacent to Flatbush Ave. and about 5 miles by water from the bridge was the site for erection of the deck spans. All material was trucked to this location for erection on the barges by a stiffleg derrick set up on land. This procedure proved to be ideal for rapid completion of the work as the barges and falsework could be used with minor modifications for one span after another. The main truss connections were riveted as far as possible before floating to position. A completely-erected span was towed to the bridge site by tug boats, where the procedure for transferring the span to the piers was similar to that followed for the through spans. The deck spans weighed from 150 to 400 tons each and required a relative movement of jacking points to water level of 2 to 3 ft. to release the barges.

As mentioned previously, most of the deck spans are of continuous construction. Some special details, therefore, were required to permit jacking and to obtain the proper stress condition in the finished structure. The jacking was provided for by connecting the spans with an auxiliary gusset at continuous points. This gusset consisted of one plate placed on the center line of the truss connecting to the bottom chord and end diagonal so as not to interfere with the permanent plates located on the outside of the members about 15 in. apart.

Proper stress conditions were assured by weighing the reactions. Taking the 3-span unit as an example, each of the 3 spans was floated to position and set on the piers as a simple span. The bottom chord joints at the continuous points were connected, leaving the top chord joints at these points open. The extreme ends of the 3-span unit were then jacked up an amount equal to the deflection of these points for zero stress in the top chords at the open joints. The connections were made for this condition, thus insuring correct reactions. As a precaution the splice plates were not fabricated until check measurements had been made.

Open mesh floor

The open grating floor used on the through spans and towers in order to reduce dead load is a source of interest. This grating, the carrying members of which are $2\frac{1}{2}\times\frac{1}{4}$ in. bars on edge and $2\frac{9}{16}$ in. on centers, is supported on transverse 7 in. I-beams spaced 16 in. on centers. The grating was delivered to the site in sections about 4 ft. wide and one panel or 38 ft. 7 in. long. These sections placed longitudinally on the bridge were riveted together to form a floor of continuously uniform pattern. The transverse joints located at each floor beam were made with shouldered rivets and a washer to provide an expansion or adjustment point. The grating was fastened by welding to the supporting 7 in. beams at $10\frac{1}{4}$ in. centers, the welds on adjacent beams being staggered.

The lift span is the longest highway lift span in the world. Complete ready for operation, it weighs 2,180 tons. The counterweights are of ordinary concrete placed in a steel box. Since the lift span was erected in the down position the counterweights were placed in the up position and utilized to raise the lift span to provide a navigation channel before floating the north or last through-span to final location.

Operation of the lift span is by electricity for both the normal and auxiliary movements. All operating machinery is located at the tower tops and synchronized electrically so that the span is always maintained level. The normal driving motors, one on each tower, are 200 hp., and the synchronizing motors are of the same size. The auxiliary motors, both driv-

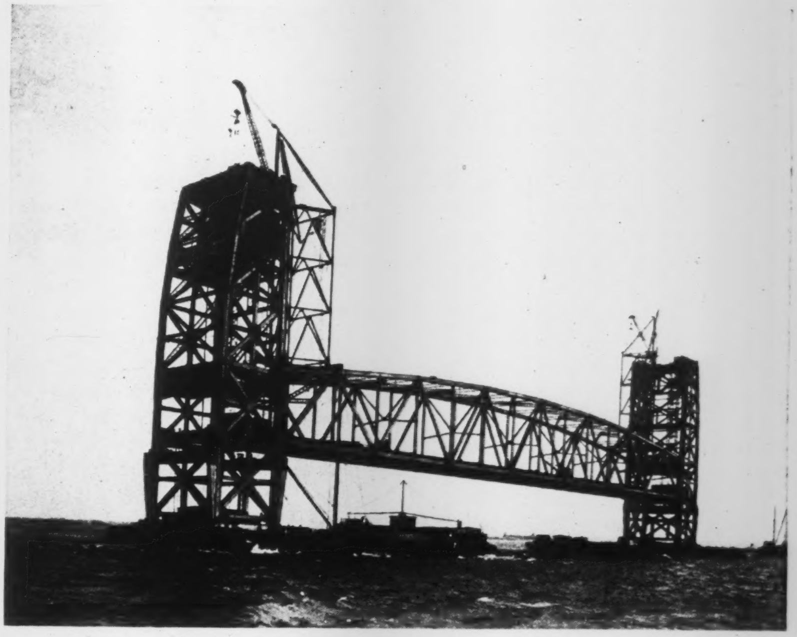


Fig. 3. The towers were raised to full height by derricks on triangular towers set on the top chord of the lift span.

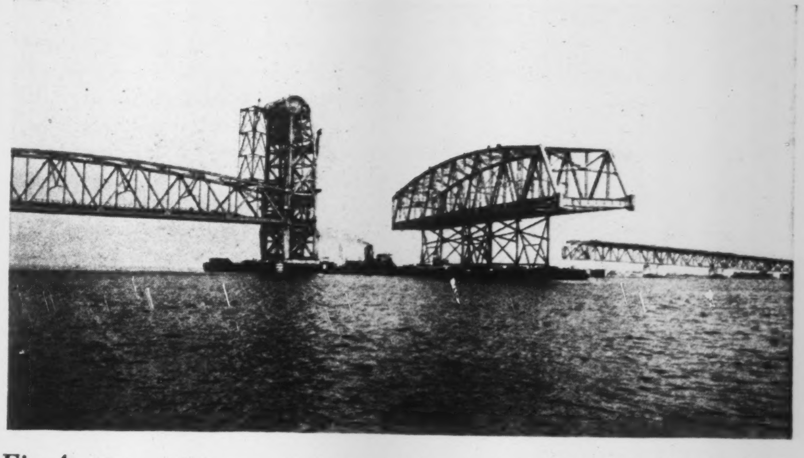


Fig. 4. Masts and booms on the tower face were used to assemble the 540-ft. fixed spans that flank each end of the lift span.

ing and synchronizing, are 50 hp. Time of operation is 2 min. for normal and 7 min. for auxiliary operation.

The architectural treatment of the tower tops is noteworthy. Plates ¼ in. thick were used for the covering which forms a housing for the machinery. The curve of the top is a form of spiral continuing the line of the rear tower columns. Fins or ribs and openings in the outer plates on the sides of the towers are used for embellishment. The structure is painted a gray-green and in order to accentuate the main lines of the towers, the counterweights, elevator shafts, operator's house and other secondary parts were painted with aluminum paint.

Robert Moses is the sole member of the Marine Parkway Authority. Madigan-Hyland were engineers in charge of construction for the Authority. The designing engineers were Waddell & Hardesty for the through spans and towers and Robinson & Steinman for the deck spans. The American Bridge Co. had the contract for the superstructure, including the concrete deck and all electrical and mechanical equipment for the lift span.

Concrete Knowledge and Practice

New facts brought out at American Concrete Institute meeting include data on mass concrete, strength of eccentrically loaded columns and rigid frames

MAJOR CONTRIBUTIONS to better concrete practice made at the meeting of the American Concrete Institute at Chicago, Feb. 22 to 24, as briefly reported last week, related to thermal cracking in mass concrete, strength of columns under eccentric load, and rigid frame tests. Other matters of practical character covered the action of curing compounds, construction methods on the San Jacinto and Edison memorial towers, and the corrosion of mortars and concretes. Theory and design were also represented.

Condensed abstracts of the principal papers, below, give a general survey of the facts brought out. For the full details of the investigations reference may be made to the forthcoming issues of the *Journal* of the institute.

Mass concrete heating and crack control

Careful examination of a large number of existing concrete dams, reported by F. R. McMillan, director of research, Portland Cement Association, Chicago, revealed wide differences in the integrity of different structures and indicated some definite conclusions on the value of construction practices.

At Crystal Springs Dam of the Spring Valley water system, San Francisco built 50 years ago, everything was favorable for mass concrete construction—low water-cement ratio, dry consistency, slow progress, small units, slow-hardening cement; the dam is virtually crack-free. In Elephant Butte and Arrowrock dams of the Bureau of Reclamation the high temperature rise which makes the problem of mass concrete so difficult was largely avoided by the active content of the sand cement of which they were built, and these dams also have few if any cracks, though they are not wholly resistant to erosion and corrosion. In more recent dams, with rates of placing greatly in excess of any of the earlier work, "success has been achieved just about in proportion to the advantage taken of the opportunities offered in the materials and workmanship to obtain watertight concrete and prevent cracking or leakage at the joints. The poorest results are those obtained where the work was carried out rapidly but with little appreciation of the importance of high temperature rise or the evils of poor workmanship." Morris and Boulder dams were found practically free from cracks, while some other western storage dams showed very extensive cracking.

J. W. Kelly, of the University of California, Berkeley, analyzed the effects of adiabatic temperature rise of the hardening concrete as modified by surface cooling, plastic flow and change of *modulus*. The results are, of course, affected by the temperature at the time of placing the concrete, the amount of cement per unit volume, and the strength of the concrete. He concluded that all factors in the case argue for lowest placing temperature. Tension due to temperature change may be reduced, Mr. Kelly concluded, by use of low-heat concrete, low temperature and favorable rate of placement, and concrete of favorable thermal, elastic and plastic properties.

Study of temperature changes in mass concrete by Prof. R. W. Carlson, Massachusetts Institute of Technology, Cambridge, was carried out for three types of cement, 0.9 bbl. per cu. yd., 70 deg. F. casting temperature, a working rate of one 5-ft. lift every four days, and 1 sq. ft. per day diffusivity of the concrete. Under these conditions the temperature rise in the interior of the mass reached 43 deg., 36 deg. and 24 deg. respectively for rapid-hardening, modified and low-heat cement. Reduction in cement content had an equivalent percentage effect in the way of temperature reduction. Thickness of lift was not as important a factor as over-all rate of construction. Each degree of precooling resulted in 0.6 deg. reduction of concrete temperature. Removal of thick side forms after two days was found to result in a serious temperature drop, going into a depth of 12 in.

A comprehensive review of the relation between concreting methods

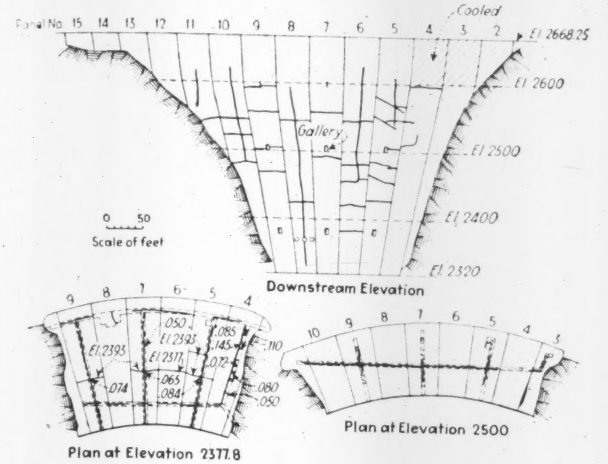


Fig. 1. Extensive cracking developed in Owyhee Dam during cooling after a 47 deg. temperature rise in the concrete.

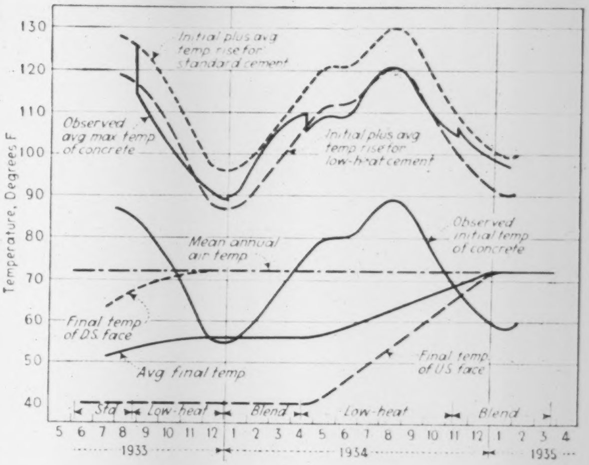


Fig. 2. Boulder Dam temperatures varied with conditions at time of placing concrete.

and cracking of a number of large dams, reported by R. F. Blanks, H. S. Meissner, and C. Rawhouser, of the Bureau of Reclamation, Denver, showed that observed cracking ranges in degree from practically none to extremely severe. Pronounced cracking difficulties have developed particularly in recent years due to rapid construction, use of rapid-hardening cement of high heat evolution, and increasing dimensions of structures.

Owyhee Dam, completed in 1932, was built fairly rapidly, with 1 bbl. per cu. yd., radial joints spaced 50 ft. apart, lifts limited to 4 ft., and very thorough grouting of joints. The maximum temperature rise found was 47 deg. The structure is stable and watertight in spite of extensive crack development.

Boulder Dam is much larger, was built more rapidly and under high temperature conditions, has 50-ft. joint spacing each way, low-heat cement, 1 bbl. per yd., 5-ft. lifts, and artificial cooling to reduce temperature stresses. Such cracks as are found are superficial and may be chargeable to drying shrinkage.

Grand Coulee, also built in 50-ft. prisms, used modified cement in the base section in the belief that the lower temperature of placing combined with artificial cooling would hold down cracking. It has cracked less than Owyhee but more than Boulder, and low-heat cement may be used for the upper section. The length of opened crack per linear foot of gallery in the three dams mentioned is 2.47, 0.72 and 1.85.

Other dams show somewhat analogous results. Morris Dam, of low-heat cement, reduced cement content and 50-ft. transverse joint spacing, has only two cracks of significance, both longitudinal. Tygart Dam, of modified cement, with transverse joints and lifts 5 to 10 ft. high, showed 35 deg. temperature rise, and in the lower part developed a major longitudinal crack midway between the faces supplemented by smaller transverse and longitudinal cracks spaced about 20 ft. apart; cracking was reduced by using lower cement content in the inner parts. Norris Dam, with 5-ft. lifts, of modified cement, has rather numerous vertical cracks, both transverse and longitudinal, but none regarded as structurally serious; temperature rise was 35 deg. Bartlett Dam, with its 7-ft. buttress walls, is being built with contraction slots 50 ft. apart, filled after maximum heat has subsided. The lower parts were built with modified cement, the upper parts with low-heat cement, which latter reduced cracking markedly. Experience at Rodriguez Dam (standard and low-heat cements) was similar. The piers and anchorages of the Bay and Golden Gate bridges were built of both modified and portland-puzzolan cement, and show less cracking in the sections built of the latter. Bonneville, built mainly of portland-puzzolan, shows less cracking than would be expected with standard or modified cement. Marshall Ford is using low-heat cement; Parker Dam uses low-heat cement and cooling.

The authors concluded that precautions to reduce cracking have yielded gratifying results.

The temperature rise in various dams using 5-ft. lifts reaches 40 deg. F. for standard and 23 deg. for low-heat cement, at 1 bbl. per yd. If the top surface is exposed only three days, however, all types of cement will reach 47 deg. after 6 to 12 months; if exposed ten days, only 32 deg. Cooling should be started at once after concrete placing, to withdraw the maximum amount of setting heat. Placing temperature must be considered, however. At Boulder Dam the maximum temperature drop was very high on this account (Fig. 2).

The general conclusion of the authors is that the most effective single method for improving cracking conditions is the use of low-heat or portland-puzzolan cement. Reduced cement content, limitation of thickness of lifts and rate and time of placement, artificial cooling, and contraction joints, are other effective measures.

Tests of columns under eccentric load

Because of the requirements of rigid-frame construction, which involves large amounts of bending of the supporting columns, more exact knowledge of the strength of reinforced concrete columns under eccentric load has become important. A series of tests on 82 columns has therefore been carried out at the University of Illinois, the results of which were reported by Prof. F. E.

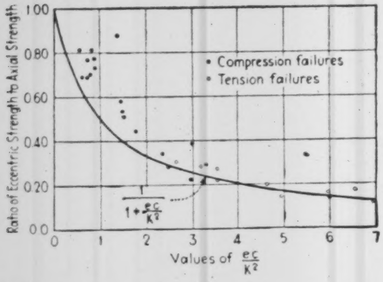


Fig. 3. Eccentrically-loaded columns had strengths closely paralleling curve of combined stress—Illinois tests of 1936

Richart and T. A. Olson. All had 8-in. core and were 5 ft. long; some were tied and some had spirals. Load was applied with eccentricities of 1 to 8 in.; check columns were tested under central load. Roller shoes and special loading brackets assured precision of test conditions. Strains and deflections were read.

All columns with 8-in. eccentricity and half of those with 4-in. eccentricity failed by tension, the others by compression. The plotted ratios of strength to that of an auxiliary loaded column (Fig. 3) were closely parallel to the curve of combined stress, $\frac{1}{1+\frac{ec}{k^2}}$. Omitting the effect of the steel, for compression failure, the curve is equivalent to $\frac{1}{1+\frac{8e}{d}}$ for circular columns and $\frac{1}{1+\frac{6e}{d}}$ for square columns, d being the over-all depth of the column. However, as the test points lie well above the curve the eccentrically loaded columns have a margin of strength attributable to non-rectilinear stress distribution, as occurs in reinforced concrete beams.

Spirally reinforced columns were found to be 5 per cent stronger than the tied columns in compression failure, but 9 per cent weaker when the failure was in tension. Columns of 4,000-lb. concrete reinforced with rail steel (yield point 63,200) were 8 to 22 per cent stronger than those reinforced with medium steel (yield point 46,700).

Earlier tests of eccentrically loaded columns are in satisfactory agreement with the present series (Withey 1910; Bach and Graf 1912-13; University of Illinois 1925).

Since the relative excess of strength of columns under eccentric loading as compared with centrally loaded columns was held to be due to the same causes that are responsible for the excess strength of beams, reflected in the higher working stresses allowed in flexure than in direct compression, the authors propose that the permissible compressive stress under eccentric loading be defined by a transition curve between the allowable axial stress and the allowable flexural stress.

Curing compounds

An extended series of tests of bituminous and other curing compounds for concrete, reported by H. S. Meissner and S. E. Smith, of the U. S. Bureau of Reclamation, Denver, indicated that the equivalent of two weeks of water curing can be produced by suitable curing compounds, when the exposure is not too severe. It is important to apply the compound before any surface moisture is lost. There is need for a good, cheap clear compound for use in arid regions in summer, as bituminous coatings, while effective, disfigure the concrete and absorb heat. Emulsions are objectionable for practical reasons; asphalt and tar cutbacks are best, as they are easily sprayed on and dry rapidly to firm, dry films. Linseed oil is too costly and colors the concrete. Paraffin is only partly effective, and cracks and peels. A composite of paraffin base wax and chinawood oil, with a quick-fading dye, was satisfactory except in places of high summer temperature.

The results confirmed those obtained by Tuthill: that coaltar cutbacks are superior to asphalt and paraffin cutbacks, and that a second coat seals the imperfections of the first. The right blend of coaltar, pitch and naphtha is important: 130 to 145 deg. F. melting-point, with acid-free coaltar naphtha of 240 to 430 deg. distillation test. The coverage (two coats) ranges for different compounds from 100 to 190 sq. ft. per gal.

Sandblast tests of cured concrete showed that to obtain a well cured surface the curing compound should be applied immediately after pouring; where application must be delayed water-curing should be used until the compound can be applied.

Thomas E. Stanton, Jr., and I. C. Meder, of the California Division of Highways, Sacramento, presented the results of sulfate durability tests of commercial cements and a few special cements. The results apply specifically to California cements, but indicate some major trends of general importance. In tests of 6x6-in. cylinders exposed to the action of alkali soil containing about 18 per cent of soluble salts (about 75 per cent sodium and 16 per cent magnesium sulfate), the most reliable index of resistance to sulfate attack, though not an absolute indicator, was the computed C_3A content of the cement; neither the C_4AF content nor the $C_4AF + C_3A$ content afforded a reliable index.

It was found that the resistance of the concrete could be increased either by reducing the C_3A content through addition of iron oxide to the raw mix, or by intergrinding a good siliceous admixture with the clinker. Durability can also be increased by increasing the cement content and simultaneously lowering the water ratio. With a given cement factor the durability can be increased by decreasing the water ratio, if this can be done without sacrificing workability.

Points in concrete design

A condensed discussion by twenty engineers on design points lying beyond code limits was reported by Albert Smith, of Smith & Brown, Engineers, Chicago. The twenty agreed that footings, columns and long girders have a larger factor of safety than minor beams and slabs, that simplicity in arrangement of steel is more important than saving a few pounds, that intricate steel should have its position shown by sketches, that it is not good practice to use minimum rod spacing, that more attention should be given to spacing of rod supports, and that construction of structural frames should be under inspection by an engineer directed by the designer.

Footing base slabs when designed for 3,000-lb. concrete by code are thinner than practical judgment approves; under conventional methods of computation lower stresses should be used. For column dowels an embedment length of 24 diameters is considered too short for intermediate grade steel in 3,000-lb. concrete; in view of defects resulting from difficult concrete placing and spacing 30 diameters is preferred as a minimum. As to offsetting vertical steel in columns, practice varies and difficulties exist in all methods. For inclined stirrups, welded stirrup nests are preferred if they can be obtained at reasonable price. Prolonging top and bottom rods as reinforcement against negative moment may help to cover the extra cost.

Contraction steel to protect against shrinkage cracking is desirable, with a minimum of ⅜ rounds 18 in. apart (cost 1c. per sq. ft.). Slab cracking and the effect of slabs on walls is a frequent source of trouble. Where a parapet rests on the concrete roof slab in a wall-bearing job the shrinkage of the slab has in many cases shortened the wall badly and leave the parapet wall projecting at the ends of the building. "We do a pretty poor job in designing for shrinkage," Mr. Smith concluded.

Prof. G. A. Maney, Northwestern University, Evanston, set forth a time-saving procedure for indeterminate frame computations. It involves calculating moments first for one end fixed, then determining the effect of this moment on the deflection and moment of the other end of the member, and proceeding progressively back and forth; residuals become negligible in about half the number of steps required under other procedures, as rapid convergence may be obtained by dividing the unknowns into alternate groups.

Rigid frame tests confirm design practice

Because rigid-frame bridge design lacks the background of experience to confirm its practices, full-sized tests were made on several rigid-frame spans at the University of Illinois by Prof. Wilbur M. Wilson and Ralph W. Kluge. They reported their results in a paper, "Tests of Rigid-Frame Bridges." The work was done under a cooperative agreement with the Portland Cement Association.

Two frames were tested, one with and one without shear reinforcement: 48 ft. between legs, 16 ft. high from base to deck, 1½ ft. wide, crown thickness 16 in., depth and width at knee 36 in., width of leg at base 24 in., reinforced with 1-in. round, medium-grade plain rods (4 bottom and 2 top rods at crown; 4 outer and 4 inner rods at leg; 5 outer and 5 inner rods at foot). The frames were designed for the equivalent of H-20 truck loading, 45 deg. temperature change either way, and a shrinkage coefficient of 0.0003. They were proportioned for either hinged or fixed-end support. The maximum design stresses reached 1,160 lb. per sq. in. in the concrete at the crown and 20,000 lb. per sq. in. in the steel at the base.

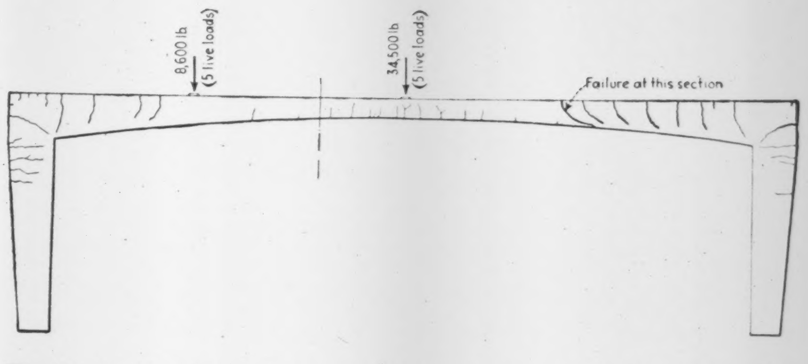


Fig. 4. Cracks just before failure of 48-ft. rigid-frame span under five times design live-load.

For the tests the base could be moved in the direction of the three components, and the reactions weighed. Influence lines were determined under a 2,000-lb. load applied at different points.

In the first frame, which had no shear reinforcement, the base moments differed widely from those computed. The second frame, built with shear reinforcement in deck and legs, showed similar difference, though more regularly distributed. However, crown and knee moments computed from the reactions checked quite closely with those calculated by elastic analysis.

Tests of cores cut from the structure showed moduli varying 10 to 20 per cent. This, however, did not account for the abnormality of the reactions, and after extensive search a fine crack was found close to the base of the leg. As a check, frames of 6-ft. span without reinforcement were tested, and here the observed base moments agreed with calculation, proving that the crack in the large structure was responsible for departure of the moments.

The general result as to elastic behavior of the frames was that the hinged-end tests checked with the results of analysis; fixed-end tests also checked closely except for base moment. It was concluded that base moment has little influence on stresses at knee and crown.

The frame without shear reinforcement was tested to destruction, under multiples of the design live load of 6,900 lb. at midspan and 1,720 lb. at a point 14 ft. east. The development of cracks under five live loads is pictured in Fig. 4. While the frame was under five live loads an additional load of 2,000 lb. was applied at the crown, and during its application the frame failed suddenly by shear near the west quarter point, very close to the line of thrust. The shear at this section under the failure load was only 72 lb. per sq. in.

At the time of failure the steel stress as shown by strain readings had passed the yield point. The concrete compression at the crown was 3,150 lb. (against 3,860 lb. cylinder strength). It was concluded that a rigid-frame bridge can develop the full strength of the concrete; that design by elastic theory for uniform modulus and uncracked section is safe and in close accord with test results; that hinged bases are advantageous for long spans and not particularly disadvantageous for short spans; that the deck and legs should have shear reinforcement even though calculated unit shear is small; and that in view of the serious effect of failure at the knee it is desirable to design the knee for a moment somewhat greater than that computed for the structure.

Mr. Kluge, adding to the paper, reported that comparative calculations for spans of 30, 40, 60 and 80 ft. showed slightly better results for hinged ends and for fixed ends, the difference being small, however. The result was held to be reassuring, as cracks are likely to develop near the base of the frame legs.

FROM FIELD AND OFFICE

Working problems and time-saving methods . . . Edited by Charles S. Hill

Soil Tamp for Proctor Test

In studying fills and stabilized base it was found necessary to devise a mechanical method for compacting soils in the Proctor mold, so as to eliminate the human element. A Tinius Olsen toughness machine was modified as described: The anvil was surmounted by a metal plate with a circular hole in it. The compactor mold was equipped with a circular base which fitted into the circular hole. This allowed the mold to be rotated while being tamped. The tamp was so constructed as to slide freely on guides and strike about ¼ in. from the edge of the mold. Since the diameter of the tamp was half the diameter of the mold the entire surface was compacted as the mold was rotated.

The tamp was raised by a rope and a pulley. The release catch was arranged so that it could be tripped at any desired height by pegs placed 1 cm. apart in a vertical support. The tamp was so constructed as to allow additional weight to be added.

Olsen toughness machine outfitted to compact soil samples in the Proctor mold.

This apparatus enabled different operators to check more accurately than was possible by the hand-tamping method. It also permitted a study of the effect of compacting soils at different pressures by varying the weight of the tamper and the height of the drop.—South Carolina State Highway Department, Columbia, S. C.

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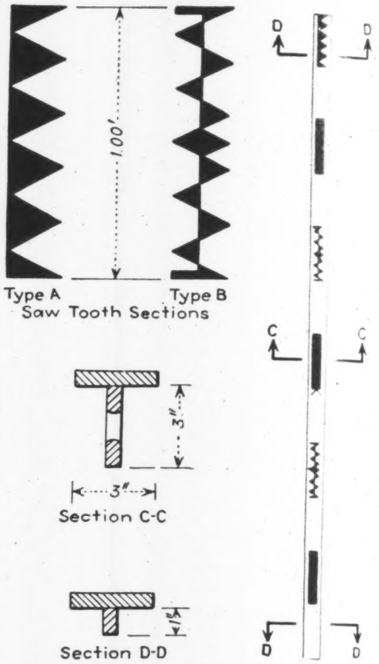
Easy Reading Stadia Rod

by HAROLD K. PALMER

Santa Monica, Calif.

Some twenty-five years ago it was suggested to the writer that the stadia rod be marked with alternate white and black blocks, each 1 ft. long, with two saw tooth sections of the *A* type, as shown in the drawing, one of them being at the top of the rod. This proved very satisfactory but it was difficult to estimate distances closer than about 2 ft. After trying several designs type *B* saw tooth was devised; with this it is possible to read to 5 ft. directly and to 1 ft. by estimation. The one drawback was that when the air began to "boil" or the distance exceeded about 500 ft. the B-saw tooth merged into a wavy line and became useless. To overcome this the rods were marked with an A-type saw tooth at the top for long-distance shots, and two B saw-tooth sections for short distances. An inverted V served to distinguish the 5-ft. marks for use in measuring vertical angles, which obviated all danger of sometimes setting the horizontal wire on the wrong foot mark.

The rod shown in the drawing was 12 ft. long and was so satisfactory it was used for several years, surviving two or three coats of paint. The shape which finally proved most satisfactory had a tee section, with a handle cut in the web to facilitate carrying. It was made of two pieces of 3¾ in. flooring with the tongues and grooves planed off and the backs planed smooth. The web was cut down to about 1 in. high (or less) at the ends, but was a full 3 in. high at the middle, where the hand hole was cut. The web was screwed securely to the back of the face board and both front and back were given two coats of white paint.



Stadia rod marking that helps out many common reading troubles.

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Colored Tags Identify Blueprints

by J. W. SILLIMAN

Resident Engineer,
California Division of Highways,
Bakersfield, Calif.

In all engineering offices, time is saved, confusion eliminated and neatness and efficiency promoted if blueprints are filed and identified by attached colored tags instead of using the usual small disk covered with tightly squeezed tiny lettering. A convenient and easily visible filing rack should be provided and all rolled prints tagged with distinct easily-identified colored tags. To one side the necessary plan index-color chart should be placed. The capacity of any one file is limited only by the ingenuity of the engineer in providing the required number of different and distinct tags. This color filing system may be extended and expanded at will, with little modification, to include filed correspondence, estimates, calculations, etc. Different color patterns may be used to separate several jobs or contracts. The colors and color patterns of frequently used drawings are soon memorized and the color-index chart is used only to identify the less frequently used plans. In actual practice this plan has proved unusually practical and well worth while.

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Curve and Grade Indicators

by Allen P. Child

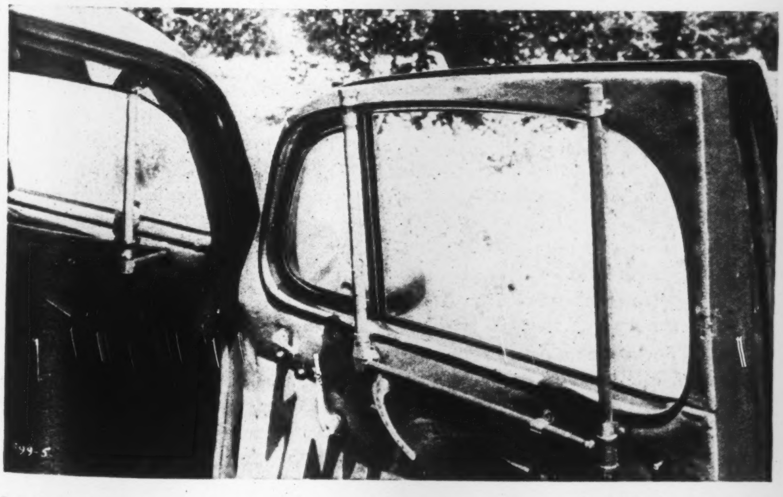
Kansas City, Mo.

In making road-inventory surveys the California division of highways is using grade and slant indicator-levels mounted in an automobile as illustrated.

The device for determining the superelevation on curves is made of a 3-ft. piece of pipe attached to the car dash in a horizontal position; to its left end a vertical pipe 15 in. long is attached and on the right end a 15-in. vertical water-gage glass. The tops of the vertical glass and pipe are capped and vented and the pipes and glass are filled with light oil to midheight of the glass. On the glass is a scale which indicates hundredths of a foot per foot. When the car is stopped on a curve the oil comes to rest and the observer notes the superelevation without the necessity of leaving the car. The instrument has been designated as a superometer.

For measuring the percentage of grades a device known as a gradometer has been devised on the same principles and for operation is attached to the inside of the right front door. Its glass gage shows the percentage of grade at all positions of the car so that it can be easily read by the observer in the car.

The writer is indebted to T. H. Dennis, Maintenance Engineer, Division of Highways, California Department of Public Works, for the information concerning the construction of these instruments. They were designed, constructed and installed by the maintenance division of the department.



Levels installed on dash and door of car simplify making records of curve superelevation and grade percentage.

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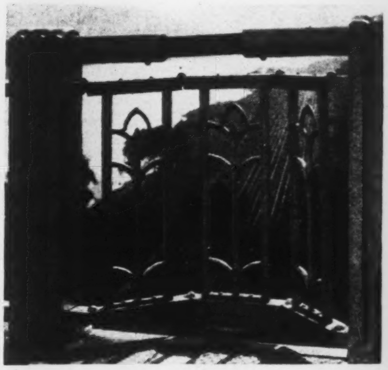

Expansion joint in bridge rail designed to allow for temperature variations of from 20 deg. below zero to 110 deg. above.

Expansion Hinge in Bridge Rail

An unusual design detail of the High Level Bridge completed last fall across the Monongahela River at Pittsburgh, Pa., is the novel expansion joint in the railing, which is designed to accommodate a maximum linear variation of 20 in. The expansion joint is composed of three hinged panels working into a recess beyond the straight line of the railing, with the pipe banister sliding on a brass bushing. Built to allow for a spread in temperature from —20 to 110 deg. F., the expansion joint is estimated to vary the recess from a minimum depth of 3 in. to a maximum of about 13 in. There are two of these expansion joints on each railing, each of them servicing a railing length up to 1,445 ft. The railing is of malleable iron.

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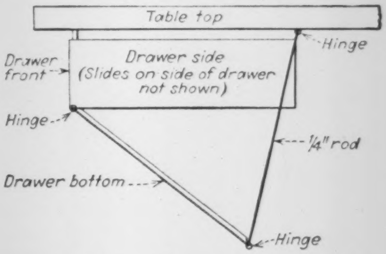
Dump Drawer for Briquet Table

by H. F. Hedderich

Superintendent Physical Laboratory, Indiana Highway Department, Indianapolis, Ind.

Briquet-making tables in a cement laboratory must be cleaned of excess mortar after each set of briquets is molded. To facilitate this cleaning and to avoid having the waste beneath the operator's feet, a simple dump drawer was designed and incorporated in the Indiana highway department cement-laboratory tables.



Dump drawer handles waste from cement briquet-molding table.

This consists of an ordinary drawer, except that the bottom is fastened only at the front edge, at which point it is hinged to permit it to drop down. Rods hinged at the back corners of the drawer bottom are also hinged to the under side of the table. When the drawer is pulled out the bottom rises to the level position. Material is then brushed or scraped from the table top into the drawer. On closing, the bottom takes a sharp slope and the material slides into a box provided for the purpose. This device has proved much more convenient and scatters less material than does a loose pan or similar arrangement.

Colorado Highway Conference

by C. L. Eckel

Head, Department of Civil Engineering, University of Colorado, Boulder, Colo.

Western state engineers offer opinions on current problems in highway design and construction practice

A ROUND-UP of opinion on current roadbuilding practice gave especial interest to the highway engineering conference held at the University of Colorado Jan. 13 and 14, 1938. This was the twelfth annual conference that has been sponsored by the university in which the highway departments of Wyoming and Colorado, and the district office of the U. S. Bureau of Public Roads have participated. In this particular meeting emphasis was given to soil stabilization and highway safety, but problems of highway construction and other topics of general interest were given attention.

Kirk A. Keegan, Statistician, National Safety Council, discussed the engineer's responsibility for making highways safe. Probably more than two-lane roads will never be economically justifiable on more than 5 per cent of our highway system. It is obviously impossible to think of rebuilding or of providing adequate safety devices for every mile of our vast two-lane mileage, but analysis of accident records will indicate locations which require particular consideration and treatment and after factual determination of need, it is the highway engineer's responsibility to provide adequate reconstruction or other safety features. In discussion, John W. Wheeler, engineer of highway negotiations of the C. B. & Q. R. R. suggested analysis of data obtained in the state-wide highway-planning surveys to determine the least-used grade crossings with the view of eliminating possibly half of those now existing.

In a paper on design standards of the Colorado state highway department, John S. Marshall, chief draftsman, discussed the effect of increased speeds on changes in highway design. Colorado standards include four types of highways for each of three classes of terrain (level or moderately rolling, hilly and mountainous). Design speeds ranging from 70 to 30 mi. per hour; as light curvature as can be obtained economically; transition spirals; aequate sight distances or increased roadway width where sight distances are inadequate; a maximum grade of 6 per cent; compensated grades if degree of curvature exceeds 10 deg.; 11-ft. traffic lanes, and increased attention to flat shoulder slopes, drainage and blending with natural terrain are current design standards.

Professor W. H. Thoman, of the University of Colorado, deprecated a tendency on the part of bridge engineers to design reinforced concrete highway structures for stresses appropriate for 2,000-lb. concrete and to specify a design mix which will yield a concrete with a 28-day strength of 3,000 lb. to 5,000 lb. per sq. in. Improved methods of manufacture of concrete and vibratory placing, with consequent uniform product merit the use of higher allowable stresses and greater ingenuity in the use of reinforced concrete in highway structure.

In discussion, W. H. Fisher, Bridge Engineer, Wyoming Highway Department, pointed out that Wyoming requires vibratory placing of concrete in all highway structures and in some instances expects to increase allowable concrete stresses to $f_c =$ 1,100 lb. per sq. in.; however he cannot foresee any great economy in this practice unless allowable stresses are also increased in the reinforcing steel. Mr. Fisher does not believe that a continuous structure, especially of the "rigid frame" type, is inherently economical and has not yet found a situation in Wyoming in which this structure has been found to be the most economical solution for a bridge crossing.

Ben M. Richardson and M. F. Maloney, Bridge Designers, Colorado State Highway Department, outlined recent developments in highway bridges of steel, concrete and timber. A tendency toward increased use of cantilever, continuous and rigid frame steel structure and of structural welding was noted. Long-span reinforced concrete structures of the girder and rigid frame type have been made possible by the use of hollow box sections. Composite wood and concrete structures of the Oregon type and of the laminated timber-concrete deck type with shear developers have found increased use.

J. R. Phillips, District Engineer, Wyoming State Highway Department, stated that a bentonite formation in the northeastern part of the state has not yielded to any of numerous treatments. Consisting of 97 per cent clay and 3 per cent silt, it is unusually dry and hard in the original condition, but air slacks on exposure. Compaction in short high fills has been unsatisfactory because of difficulty in getting equipment down to the bottom of the fill. Present practice is to bulldoze this material into the gulch from above, spread with carry-all scrapers and roll at right angles to the center line until a sufficient height is obtained to permit equipment to work parallel to the center line. Slopes in sandy regions may be plated with clay to reduce wind erosion. This is satisfactory for shallow fills but difficult and expensive on high fills. Asphaltic treatment has been used.

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Vallecito Dam Is Awarded

CONTRACT for construction of Vallecito Dam, major structure in the Pine River federal reclamation project, has been awarded the Martin Wunderlich Co. of Jefferson City, Mo. on its bid of $2,116,000, the lowest of nine. Although the contract will be awarded at once, start of work will be delayed pending the approval of a repayment contract with the Pine River Irrigation District and of a memorandum of understanding between the Bureau of Indian Affairs and the Bureau of Reclamation.

Vallecito Dam will be an earth structure about 125 ft. high stretching 4,000 ft. across the Pine River above Bayfield, Colo. It will create a 126,000-acre-ft. reservoir to irrigate 69,000 acres of white and Indian lands downstream.

THE WEEK'S EVENTS

(Continued from page 355)

CALENDAR OF MEETINGS

NATIONAL SOCIETIES

American Railway Engineering Association, annual meeting, Chicago, March 15-17.

Building Officials Conference of America, annual meeting, Willard Hotel, Washington, March 21-25.

American Society of Civil Engineers, Spring meeting, Jacksonville, Fla., April 20-23.

American Shore & Beach Preservation Association, annual meeting, Washington, D. C., April 19, and Jacksonville, Fla., April 21.

American Water Works Association, annual convention, Roosevelt Hotel, New Orleans, April 24-28.

REGIONAL AND LOCAL

Connecticut Society of Civil Engineers, annual convention, Hartford, March 15-16.

New Jersey Mosquito Extermination Association, annual meeting, Hotel Chalfonte-Haddon Hall, Atlantic City, March 23-25.

Society for the Promotion of Engineering Education, regional surveying and geodesy meeting, Rensselaer Polytechnic Institute, Troy, N. Y., March 24-26.

Michigan Sewage Works Operators Association, annual conference, Lansing, Mar. 31-April 1.

Michigan Engineering Society, annual meeting, Grand Rapids, April 7-9.

Greater New York Safety Council, annual convention, Hotel Astor, New York City, April 19-21.

American Society of Civil Engineers, Texas Section, spring meeting, Texas State Hotel, Houston, April 29-30.

Section Meetings, American Water Works Association:
New York, Jamestown, Mar. 17-18.
Ky.-Tenn., Louisville, March 21-23.
Canadian, Windsor, Ont., Mar. 23-25.
Illinois, Decatur, April 5-6.
Indiana, West Lafayette, April 7-8.
Pacific Northwest, Spokane, May 19-21.

ELECTIONS AND ACTIVITIES

W. L. Maloney has been elected president of the Associated Engineers of Spokane at Spokane, Wash. The association is composed of local sections of the four founder societies and of the American Institute of Architects. Other officers are: vice-presidents, George A. Pehrson, Roy H. Clarke, H. B. Hodgins, and H. J. MacCamey; secretary-treasurer, H. C. Bender.

New officers elected at the annual meeting of the Associated Equipment Distributors were as follows: A. F. Sersanous, president; Victor L. Phillips, R. R. Nixon, vice-presidents; A. C. Blaisdell, secretary-treasurer.

Newly elected officers of the Engineers' Society of St. Paul are: president, S. E. Blunt; vice-president, W. S. Cockroft; secretary-treasurer, W. G. Ramlow; member of government, Paul Beinhorn; federation representative, J. R. Johnston.

Major A. B. Jones was elected president of the Duluth Section of the American Society of Civil Engineers at its annual meeting. Ralph Palmer and W. H. Woodbury are the new vice-presidents.

Carl T. Bowen was elected president of the Michigan Association of Road Commissioners and Engineers at the 24th annual highway conference last month at Ann Arbor. Other officers are: Leon Luke, vice-president, and L. F. Levin, secretary-treasurer.

The new officers of the District of Columbia Society of Professional Engineers are: president, Virgil C. Wagner; vice-president, Joseph Barnett; treasurer, M. C. Steuber; secretary, William L. Bach.

Clyde Walb has been named temporary chairman of a newly organized chapter at Fort Wayne of the Indiana Society of Professional Engineers. Other temporary officers are Harry K. Gottschalk, vice-chairman; C. R. McAnlis, treasurer; F. B. Mendenhall, secretary.

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Examinations, Short Courses

Iowa: Examination for professional engineering, Mar. 22-23, and for land surveying, Mar. 24, Des Moines.

Cornell University, short course for sewage works operators of New York State, March 14-26.

BRIEF NEWS

A radio program originating along the Colorado Aqueduct was broadcast Feb. 26 by the Columbia Broadcasting System. Six announcers at various points along the line of the aqueduct described the construction operations.

Salvage operations on the fallen Falls View Bridge over the Niagara River were abandoned on the Canadian side Feb. 22 by the Robertson Construction Co. Ltd., which has received title to the bridge from the International Railway Co. The section closest to the Canadian shore has been completely removed, and to proceed farther it would be necessary to knock down girders which stand 60 ft. above the ice, an operation which was felt to be too dangerous. The company has salvaged 350 tons of steel, 40,000 ft. of lumber, and 15,000 tons of copper cable. Operations may be shifted to the American side later.

Notice that a sewage disposal plant must be put into operation by Jan. 1, 1940, was given to the town of Tonawanda, N. Y. Feb. 14 by the New York State Department of Health. Unless the city moves to build a plant, the department threatens to act under the public health law and prevent the city from dumping its sewage into the Niagara River.

Highway workers in Pennsylvania are to wear red reflectors to protect them at night. Belts and shoulder straps studded with bits of red glass have been provided for 750 men, and if they prove successful 5,000 of the belts will be obtained.

Electrification of railroads of the Soviet Union is continuing rapidly, according to reports reaching this country. More than 275 miles of line were converted last year, and this year it is planned to electrify the 115-mile line from Apatity to Murmansk and to complete the Moscow suburban network which has been under construction for some time. It is intended to add Kiev to the suburban network next year. The lines are using high speed locomotives of the PB type capable of developing 75 miles per hour.

The general plan for the proposed pontoon bridge across Lake Washington at Seattle was approved Feb. 26 by the board of consulting engineers studying the project, as was the plan of financing the project by a bond issue to be retired from tolls. The board ordered immediate preparation of plans and specifications and voted to submit the project to the War Department for its approval.

OBITUARY

Charles F. Fisher, superintendent of parks of Akron, Ohio, died in Butler, Pa., Feb. 28 while on a leave of absence due to ill health. Mr. Fisher was for ten years city planning engineer at Akron, later directed a WPA traffic survey there, and had held his position as superintendent of parks for the past two years. Before his service in Akron, Mr. Fisher was street extension engineer at Portland, Ore., and secretary of the Portland city planning commission. His work at Akron was interrupted for a time while he made zoning studies of the city of Providence, R. I.

Adam Byerly, Seattle construction engineer who had been in charge of construction of many buildings and much public works in the Seattle area, died Feb. 22 at the age of 85. He had been retired from active business for more than ten years.

Luther Y. Kerr, a civil engineer active for many years in the construction of the levee works on the Mississippi River, died Feb. 23 in Memphis at the age of 72. He had retired in 1936 from his position as principal engineer in the Memphis District of the U. S. Engineers after 46 years in the government service. He studied civil engineering at the University of Missouri and in 1890 he went to work for the government as a rodman for the Missouri River Commission. A year later he was transferred to the Memphis district of the Mississippi River Commission and thereafter was engaged continuously on federal work on the Mississippi except for two years on furlough and six months in charge of construction of a waterworks plant for Greenville, Miss.

Thomas McOwen, assistant city engineer of Salt Lake City, died there Feb. 21 at the age of 60.

William Snyder, who retired five years ago from his position as construction engineer for the Georgia Marble Co. which he had held for 20 years, died in Atlanta Feb. 20 at the age of 76.

Alfredo C. Janni, consulting civil engineer, died Feb. 25 in New York City at the age of 68. Mr. Janni, a native of Italy, graduated from the University of Naples in 1893 and taught there for several years. He practiced for about 8 years in Naples. He came to this country in 1904 and shortly afterward became a structural engineer for the city of St. Louis; he entered private practice in St. Louis in 1912. Mr. Janni was the originator of the plan for constructing concrete ships which was used by the U. S. Shipping Board during the war and he was a frequent contributor to periodicals.

Wendell J. Wilson, former engineer of the Port of Seattle, died there Feb. 27 at the age of 58. Mr. Wilson was at one time in charge of railway construction for the Panama government.

Mose Emerson, 81, former county engineer of Kittitas County, Wash., died Jan. 20.

Edward J. McQuaid, 76, retired Seattle paving contractor, died there Feb. 19.

Thomas P. Egan, head of the Egan Construction Co., general contracting firm, and formerly supervisor of the Cleveland sewer department, died there recently at the age of 65.

Samuel P. West, construction engineer and founder of the S. P. West Construction Co., died recently in Windsor, Ont., at the age of 66.

PERSONAL

Frank Mumford has been appointed county engineer of York County, Nebr.

Sherwin F. Kelly has resigned his connection with Combined Geophysical Methods, Inc., of New York City, which is now ceasing business, and has taken over sole control of Geophysical Explorations, Ltd., of Toronto, as president of the company. Marion Kelly is secretary-treasurer and L. B. Wright is vice-president. The company, which has acquired the files and most of the equipment of Combined Geophysical Methods will carry out geological and geophysical surveys in Canada, the United States, and elsewhere, operating from its Toronto headquarters.

W. W. Mitchell, former assistant engineer for the Arkansas state highway department, has been named director of that department, succeeding James R. Ryan, who recently left the post. Lately, Mr. Mitchell has been an engineer-inspector in the state PWA office.

Park Swain, for the past seven years district highway maintenance engineer at McKenzie, Tenn., has become West Tennessee divisional engineer at Jackson.

W. B. Haynes, resident engineer for the Tennessee highway department at Huntsville, has been made district highway maintenance engineer at McKenzie, succeeding Park Swain.

John F. Plow, for some years assistant to the secretary of the Engineering Institute of Canada, has resigned to take a position with Charles Warnock & Co. Ltd., Montreal testing engineers.

Louis R. Howson, member of the Chicago consulting firm of Alvord, Burdick & Howson, has been appointed a member of the Illinois Board of Natural Resources and Conservation in the Department of Registration and Education.

John P. Riley has been appointed borough superintendent of Queens in the New York City Department of Housing and Building. Mr. Riley was formerly connected with the civil engineering firm of Parsons, Klapp, Brinckerhoff & Douglass.

Samuel H. Mackenzie, superintendent of the Southington, Conn., water department, has resigned his position. Mr. Mackenzie began with the Southington Water Co. in 1900 as engineer and general manager, and when the town purchased the water company in 1911 Mr. Mackenzie became superintendent.

H. G. Groves, who resigned as state materials engineer of North Dakota on Jan. 31, has taken a position with the Yale Oil Corp. of Billings, Mont.

Keith Boyd, division highway engineer at Grand Forks, N. D., has been appointed state materials engineer.

N. W. Hicks, former division engineer and acting chief engineer of the Montana Highway Department, has been appointed construction engineer for the North Dakota Highway Department. Mr. Hicks has been with the Montana department for nine years.

Carl D. Hill, chief engineer of the Pittsburgh Department of City Planning, has been appointed executive director of the Pittsburgh Housing Authority.

Ferdinand Krieger, chief engineer of the waterworks pumping stations in the Milwaukee water system, has resigned at the age of 84 after 53 years service as an employee of the city water department.

R. R. Litehiser, chief engineer of tests of the Ohio State Highway Department, will become director of engineering of the New York State Crushed Stone Association at Albany, on March 1.

Bogue Vandercook, city engineer of Longview, Wash., has resigned that position because of ill health.

Jack Jones, for several months acting city engineer of Longview, Wash., has become city engineer, succeeding Bogue Vandercook.